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for Public Comment

OFFICE OF SOLID WASTE AND EMERGENCY RESPONSE
U.S. ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

Hazardous Waste Engineering Research Laboratory
Office of Research and Development
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Cincinnati, Ohio 45268

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This Technical Resource Document is compilation of all of the available information on the design, construction, and evaluation of clay liners for waste landfills, surface impoundments, and wastepiles. The information was obtained from the literature-and from in-depth interviews with design and construction engineers and other knowledgeable individuals in both the private and government sectors. As a consequence, some information is presented for the first time in this document. The broad topics covered are: clays, with emphasis on their composition, fabric, and hydraulic conductivity; geotechnical test methods and soil properties including index properties, soil classification, and hydraulic conductivity testing; clay chemical compatibility, including a discussion of the mechanisms of interaction and a comprehensive compilation of exsisting test data from the literature and private sources; construction and quality assurance; clay liner failure mechanisms; the performance of existing clay liners based on case studies of 17 sites; and clay liner transit time prediction methods featuring an in-depth discussion of many available techniques and models.

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This report was prepared by Dr. L. J. Goldman, R. S. Truesdale, G. L. Kingsbury, C. M. Northeim, and A. S. Damle of the Research Triangle Institute (RTI), Research Triangle Park, NC, under Contract Number 68-03-3149-1-2. The U.S. Environmental Protection Agency (EPA) Project Officer was M. H. Roulier of the Hazardous Waste Engineering Research Laboratory, Cincinnati, Ohio.

This is a draft technical resource document that is being distributed by EPA for comment on the accuracy and usefulness of the information it contains. The report has received extensive technical review, but the Agency's peer and administrative review process has not yet been completed. Therefore, it does not necessarily reflect the views or policies of the Agency. Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

FOREWORD

The Environmental Protection Agency was created because of increasing public and governmental concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of the environment and the interplay of its components require a concentrated and integrated attack on the problem.

The Office of Solid Waste is responsible for issuing regulations and guidelines on the proper treatment, storage, and disposal of hazardous wastes to protect human health and the environment from the potential harm associated with improper management of these wastes. These regulations are supplemented by guidance manuals, technical guidelines, and technical resource documents, made available to assist the regulated community and facility designers in understanding the scope of the regulatory program. Publications like this one provide facility designers with state-of-the-art information on design and performance evaluation techniques.

This technical resource document is a compilation of all of the available information relevant to the design, construction, and performance of clay-lined waste management facilities. The broad topics covered are: clays; geotechnical testing of soils; the compatibility of clays and chemical wastes; the design, construction, and construction quality assurance of clay liners; potential failure mechanisms; the performance of existing facilities; and methods for predicting the useful life (transit time) based on the modeling of leachate flow through soils.

Marcia Williams, Director
Office of Solid Waste
U.S. Environmental Protection Agency

PREFACE

Subtitle C of the Resource Conservation and Recovery Act (RCRA) requires the U.S. Environmental Protection Agency (EPA) to establish a Federal hazardous waste management program. This program must ensure that hazardous wastes are handled safely from generation until final disposition. EPA issued a series of hazardous waste regulations under Subtitle C of RCRA that are published in 40 Code of Federal Regulations (CFR) 260 through 265 and 122 through 124.

Parts 264 and 265 of 40 CFR contain standards applicable to owners and operators of all facilities that treat, store, or dispose of hazardous wastes. Wastes are identified or listed as hazardous under 40 CFR Part 261. Part 264 standards are implemented through permits issued by authorized States or EPA according to 40 CFR Part 122 and Part 124 regulations. Land treatment, storage, and disposal (LTSD) regulations in 40 CFR Part 264 issued on July 26, 1982, establish performance standards for hazardous waste landfills, surface impoundments, land treatment units, and waste piles.

EPA is developing three types of documents for preparers and reviewers of permit applications for hazardous waste LTSD facilities. These types include RCRA Technical Guidance Documents, Permit Guidance Manuals, and Technical Resource Documents (TRD's).

The RCRA Technical Guidance Documents present design and operating specifications or design evaluation techniques that generally comply with or demonstrate compliance with the Design and Operating Requirements and the Closure and Post-Closure Requirements of Part 264.

The Permit Guidance Manuals are being developed to describe the permit application information the Agency seeks and to provide guidance to applicants and permit writers in addressing information requirements. These manuals will include a discussion of each step in the permitting process and a description of each set of specifications that must be considered for inclusion in the permit.

The Technical Resource Documents present state-of-the-art summaries of technologies and evaluation techniques determined by the Agency to constitute good engineering designs, practices, and procedures. They support the RCRA Technical Guidance Documents and Permit Guidance Manuals in certain areas (i.e., liners, leachate management, closure covers, and water balance) by describing current technologies and methods for designing hazardous waste facilities or for evaluating the performance of a facility design. Although emphasis is given to hazardous waste facilities, the information presented in these TRD's may be used for designing and operating nonhazardous waste LTSD facilities as well. Whereas the RCRA Technical Guidance Documents and Permit Guidance Manuals are directly related to the regulations, the information in these TRD's covers a broader perspective and should not be used to interpret the requirements of the regulations.

This document is a first edition draft being made available for public review and comment. It has undergone review by recognized experts in the technical areas covered, but Agency peer review processing has not yet been completed. Public comment is desired on the accuracy and usefulness of the information presented in this document. Comments received will be evaluated, and suggestions for improvement will be incorporated, wherever feasible, before publication of the second edition. Communications should be addressed to Docket Clerk, Office of Solid Waste (WH-562), U.S. Environmental Protection Agency, 401 M Street, S.W., Washington, D.C., 20460. The document under discussion should be identified by title and number; e.g., "Design, Construction, and Evaluation of Clay Liners for Waste Management Facilities," (EPA/530-SW-86-007).

ABSTRACT

This Technical Resource Document is a compilation of all of the available information on the design, construction, and evaluation of clay liners for waste landfills, surface impoundments, and wastepiles. The information was obtained from the literature and from in-depth interviews with design and construction engineers and other knowledgeable individuals in both the private and government sectors. As a consequence, some information is presented for the first time in this document. The broad topics covered are: clays, with emphasis on their composition, fabric, and hydraulic conductivity; geotechnical test methods and soil properties including index properties, soil classification, and hydraulic conductivity testing; clay chemical compatibility, including a discussion of the mechanisms of interaction and a comprehensive compilation of existing test data from the literature and private sources; construction and quality assurance; clay liner failure mechanisms; the performance of existing clay liners based on case studies of 17 sites; and clay liner transit time prediction methods featuring an in-depth discussion of many available techniques and models.

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CHAPTER 1

INTRODUCTION

A clay liner consists of one or more layers of cohesive soil that have been compacted to achieve a low permeability. The purpose of a clay liner in a waste management facility (landfill, waste pile, or surface impoundment) is to serve as a barrier between waste materials and the hydrogeologic environment by limiting seepage from the facility and to provide support for overlying facility components.

A clay liner is usually constructed of native soil that contains appreciable amounts of clay-sized particles; in some cases other materials, such as bentonite, are mixed with the soil when it does not contain sufficient clay. A clay liner may be overlain by one or more flexible membrane liners and primary and secondary leachate collection (drain) systems. Figure 1-1 illustrates one possible liner configuration for a surface impoundment.

A compacted clay soil liner, because of its low permeability (hydraulic conductivity), limits the steady-state seepage from a facility. In waste management facilities the clay liner also is designed to delay the beginning of seepage for the longest possible time (transit time) and to have sufficient structural stability to support itself and other facility components that may lie above it. Clay liners are used not only in waste management facilities but also in many other applications such as water storage and conveyance structures.

Although clay liners are a widely used technology, there is very little information on their performance in limiting seepage from operating waste facilities. [7.2.1; 7.3.2]* Performance predictions have been based primarily on results from permeability tests conducted on laboratory-compacted soils. Laboratory testing has provided a great deal of data that demonstrate the permeability (hydraulic conductivity) that can be achieved with various soil materials and compaction methods and how various types of wastes can interact with soil materials to change this permeability. Even though it is common practice to use such laboratory results to predict the field behavior of a liner, there are few field data that confirm the validity of laboratory-derived soil permeability values as a measure of field permeability. There are some field data showing that permeability and seepage rates for field-compacted clay soil liners are much greater than for laboratory-compacted soils. [2.3.2; 3.8.3.1] Until more field data are available on the performance of clay liners constructed with the best current technology, permeabilities of laboratory-compacted soils should be

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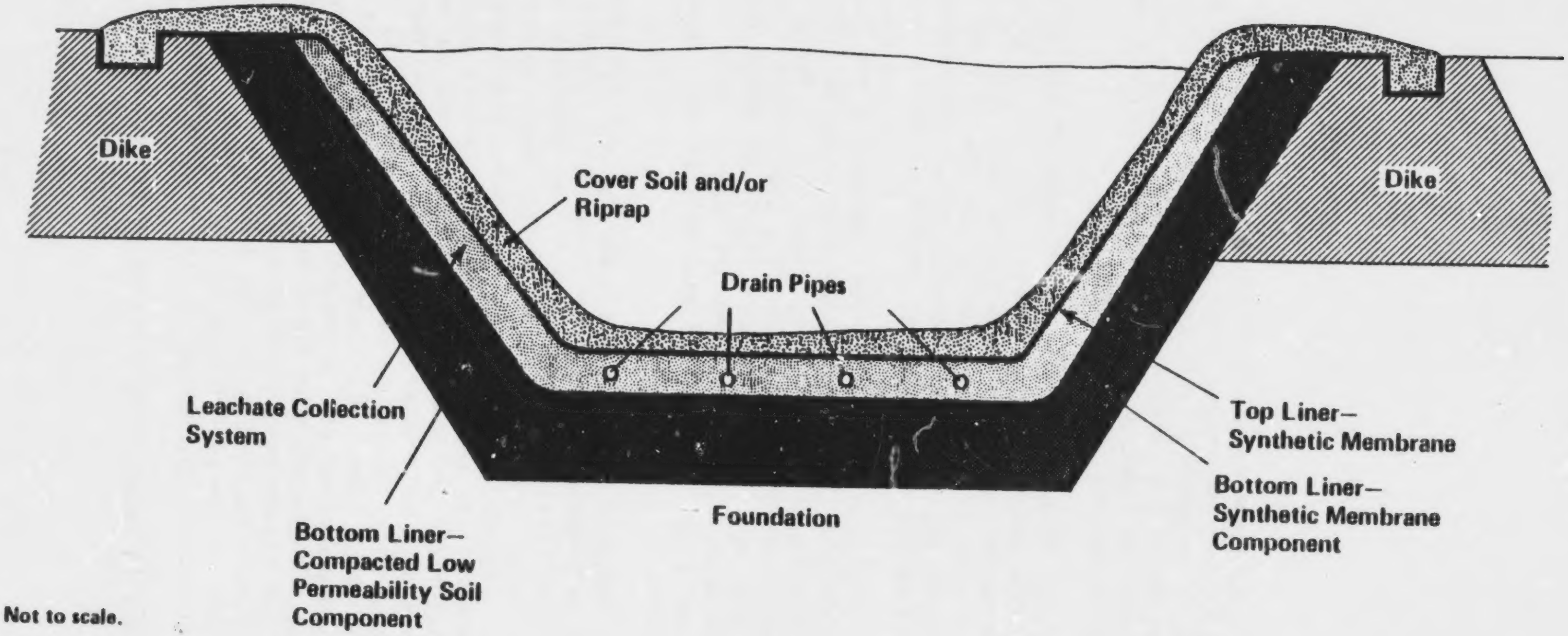
*Numbers in brackets refer to relevant sections of this document.


Dike
Cover Soil and/or Riprap
Drain Pipes
Dike
Leachate Collection System
Top Liner—Synthetic Membrane
Bottom Liner—Synthetic Membrane Component
Foundation
Bottom Liner—Compacted Low Permeability Soil Component

Not to scale.

Figure 1-1. Liner configuration for surface impoundment.

regarded as a goal rather than an accurate estimate of the performance of field-constructed clay liners.

1.1 SCOPE

The objective of this Technical Resource Document (TRD) is to provide, in a single source, all of the available information on design, construction, and performance of clay-lined waste management facilities. The broad topics covered are: clay properties and characteristics [2.0]; geotechnical testing of soils [3.0]; the compatibility of clays and chemical wastes [4.0]; the design, construction, and construction quality assurance of clay liners [5.0]; potential failure mechanisms [6.0]; the performance of existing facilities [7.0]; and methods of predicting the useful life (transit time) based on the modeling of leachate flow through soils [8.0]. It should be noted that the design and construction section [5.0] is limited to practices currently in use by engineers. Information on recommended practices is available in guidance documents issued by the USEPA Office of Solid Waste, Washington, D.C.

One of the major sources of information for this document was a series of in-depth interviews with over 30 design/construction engineers who have hands-on experience with clay-lined facilities in all areas of the country. Some of the material obtained in these interviews appears here, in print, for the first time. Opinions and preferences stated in this document, except when referenced to published material, were obtained from these interviews and do not necessarily represent the preferences of the authors or USEPA.

This document contains some soils information taken from a Technical Resource Document on flexible membrane liners, Lining of Waste Impoundment and Disposal Facilities (Haxo, 1983). Other important sources of information were the published literature, discussions with researchers, interviews with personnel from waste management companies and industries, and contacts with State and Federal regulatory personnel. Waste management companies and regulatory agencies were the primary sources for the information contained in Chapter 7, Clay Liner Performance.

1.2 SUMMARY OF CURRENT PRACTICES

The following is a summary of major current practices in clay liner construction that are believed to affect the permeability of the liner; other aspects of earthwork (cut and fill volumes, etc.) are not covered. The practices summarized are average or typical of a large number of the cases observed by the authors at operating sites or in construction documentation. Instances of better or poorer practices were also frequently encountered by the authors.

1.2.1 Investigation of Site Conditions (Field)

Site investigations are conducted before construction to identify and investigate borrow areas for liner material. The borrow area and the soils that will underly the liner are investigated through borings, pits, and trenches cut across the area. In situ soil properties and groundwater conditions are identified and taken into account in the design to avoid structural problems such as hydraulic uplift (heaving) or settlement of the soil underlying the liner. The foundation (soil) underlying the proposed facility site is evaluated for structural properties through laboratory tests on removed samples and in situ testing. [5.1.1; 5.3.3.2.1]

1.2.2 Material Selection and Characterization (Laboratory)

1.2.2.1 Foundation Soil--

Soil underlying the proposed facility must possess bearing and shear strength adequate to support expected loading. If tests on samples examined in the laboratory and onsite bearing tests show inadequate properties, the site design and construction plans will include specifications that provide for excavation and recompaction of an adequate foundation. [5.1.3.2.1; 6.2; 6.3].

1.2.2.2 Index Properties of Liner Materials--

Samples of soils from potential borrow areas are analyzed to determine their index properties (grain size distribution, Atterberg limits, and Unified Soil Classification). [3.3; 3.5.2; A-26; A-28; A-29; A-34; A-37] This information is used to identify and reject undesirable materials (i.e., low plasticity index or low fines content) and to select desirable materials.

1.2.2.3 Engineering Properties of Liner Material--

A very important criterion for a suitable liner soil is whether it can be compacted so that the permeability (hydraulic conductivity), as measured in the laboratory, is 10^{-7} cm/s or less. A number of engineering firms and regulatory agencies have guidelines for the plasticity index and amount of fines (amount passing a No. 200 sieve) and some have acceptance criteria that consider only plasticity index or fines, but, at present, permeability appears to be the most heavily weighted criterion.

Selection of a liner material that can be compacted to the required permeability involves a series of laboratory tests of the engineering properties of the candidate materials. A moisture content/density relationship is established for the material by compacting samples of the material at various moisture contents with a set compactive effort. [2.3.2; 3.4.1; 3.8.4.5; A-45; A-47; A-50] The optimum molding water content is the water content at which the maximum dry density is achieved. The permeability (hydraulic conductivity) of a sample of the soil compacted with the same compactive effort at a water content slightly greater than optimum is measured. [2.3.2; 3.4.2; 3.8] The relationship between density, moisture content, compactive effort, and permeability derived from these data will be used to develop construction specifications and, for quality control of the construction process, to ensure that the water content is slightly wetter than optimum and that compaction has been sufficient to achieve the desired permeability. [3.4.2; 3.6; 5.1.4; 5.3.3.2.4] It is also necessary to construct a test fill of the liner material prior to construction to confirm that the laboratory-derived relationship can be achieved in the field. [1.2.4; 5.2.2.1.4; 5.3.3.1]

In addition to meeting permeability requirements, liner material must be compatible with the waste it is meant to contain. A laboratory-compacted sample of the soil will be tested to determine whether wastes or leachates from the proposed facility will react with it to increase its permeability and decrease its ability to control seepage from the facility. [4.3; 4.4; 7.6.4]

1.2.3 Develop Liner Design/Construction Plans

1.2.3.1 Lift and Liner Thickness--

The depth of loose soil laid down for compaction (lift thickness) is selected so that the equipment and operating conditions will be capable of imparting the required compactive effort throughout the lift. [5.2.2.1.1] Minimum liner thickness is usually set by State or Federal regulations. Beyond these minimums, liner thickness may be established on the basis of the time required for liquid or waste to pass through the liner (transit or containment time). [5.1.3.3; 7.7; 8.0] Greater than average liner thickness may be specified at critical points such as the junction of the bottom and side and below leachate collection lines or sumps. [5.2.2.1.1]

1.2.3.2 Compactive Effort--

Equipment and operating conditions (weight, number of passes, etc.) are selected in conjunction with lift thickness to ensure that the same compactive effort will be applied in the field as was used in the laboratory to achieve the specified permeability. [3.4; 5.2.2.1.4] This selection will be confirmed in the test fill. [1.2.4]

1.2.3.3 Adjust Design to Avoid Failure Mechanisms--

The design of the liner and the foundation (underlying soil) will be adjusted to avoid common failure mechanisms, as appropriate:

- Erosion [5.1.3.4; 6.6.2]
- Freezing [5.1.3.4.3; 5.2.2.3.2; 6.7]
- Piping [5.1.3.4.4; 6.4.3]
- Slope instability [6.2]
- Settlement [6.3]
- Earthquake [5.1.3.4.6; 6.8]
- Scouring (surface impoundments) [5.1.3.4.2; 6.9]
- Hydraulic uplift (heaving) [5.1.3.4.7; 7.4].

1.2.4 Pilot Construction Test (Test Fill)

A small-scale construction test is conducted to determine whether the selected combination of lift thickness, equipment, operational procedures, moisture content, and density will result in the design permeability identified during the laboratory testing. [5.2.2.1.4; 5.3.3.1]

1.2.5 Construction

1.2.5.1 Foundation Preparation--

The native soil is the foundation for the clay liner. The soil surface will be, at a minimum, cleared, grubbed, stripped, and cleaned of organic or otherwise deleterious material prior to placement of the liner material. Resulting holes and depressions in the soil or underlying rock will be filled. Where necessary, the soil may be excavated and recompacted using standard earthmoving equipment. Water content and compactive effort will be adjusted to give a firm surface for construction of the liner. The surface may be proof-rolled to locate soft spots prior to placing liner material. [5.2.1.1; 5.3.3.2.1]

1.2.5.2 Liner Material Preparation--

Soil material for the liner will be either excavated on site or brought from a nearby borrow area. Areas containing material that is unsuitable because the fines content is too low or because it contains rocks, organic matter, etc., will be identified; material from these areas will be segregated from the liner material and not used in the liner. [5.3.3.1; 7.5.1.2]

If the material is too dry or heterogeneous, construction equipment will be used to mix it and the water content may be adjusted by addition of water prior to placement. Adequate curing time must be allowed to enable added water to penetrate all liner material uniformly. Stockpiles of liner materials are often covered or seal-rolled to retard drying or erosion. [5.2.2.1]

1.2.5.3 Material Placement--

Liner material will be hauled to the site from a stockpile or borrow area and spread to the specified loose lift thickness as estimated with a shovel blade or staff. [5.2.2.1.1] Rocks and other foreign material may be removed if apparent. Clods that are larger than the lift thickness may be reduced in size with a disc or harrow. [5.2.2.1.2; 5.3.3.2.2]

Water content of the soil will have been measured on samples from the emplaced material. [3.6.2.2; A-6; A-7] If water content is less than specified in the design. water will be added by spraying from a truck or large hose before the soil is compacted. Adequate curing time must be allowed. If the soil is too wet, it will be allowed to dry somewhat before compaction. [5.2.2.1.3; 5.3.3.2.3]

1.2.5.4 Compaction--

The soil is compacted using equipment such as sheeps-foot or rubber-tired rollers until the density of the soil, at the specified moisture content, has reached the value specified in the design. [5.2.2.1.4; 5.3.3.2.4] During construction, density may be measured by direct methods such as the sand cone or balloon but is more commonly measured with a nuclear density gauge. [3.6.2.1; 5.3.3.2.4; 5.3.3.2.5; A-19; A-20; A-22] Samples of soil may be taken during compaction to determine if the water content is close enough to the design value. Alternatively, a nuclear moisture gauge may be used.

The top of a lift (layer of soil) may be scarified with a harrow or other equipment so that there will be an adequate bond with the lift placed above it. The edges of lifts are often beveled or overlapped to ensure complete coverage. [5.1.3.3; 5.2.2.1.1]

1.2.5.5 Completion--

Upon completion of the liner, samples may be taken for laboratory permeability measurements; alternatively, field permeability measurements may be made on the completed liner.

The surface of the liner may be proof-rolled to seal it and prevent erosion and the liner may be covered with a protective layer of soil to reduce desiccation cracking if there will be considerable time before the liner is placed in service or before a leachate collection system or flexible membrane liner is placed over the clay liner. [5.1.3.4.5; 5.2.2.3.1; 6.1.3]

1.3 ANALYSIS OF CURRENT PRACTICES

The current practices discussed in this section are those that would be most likely to improve clay liner performance (lower permeability) if they were changed. Most of this analysis is based on laboratory studies of the behavior of compacted soils; few field studies of the relationship between construction practice and permeability have been conducted.

1.3.1 Liner Material

Changes in the amount of fines (material passing a No. 200 sieve) in a soil strongly affect the relationship between compactive effort and water content and the maximum density and permeability that can be achieved. Consequently, efforts in the following areas are likely to result in a liner with a more uniform and lower permeability:

- Identification of the range of soil properties in the borrow area
- Determination of the water content, compactive effort, density, and permeability relationships for all the major and significantly different bodies of soil in the borrow area, and verifying these relationships in a field-compacted test fill
- Sorting soils so that those delivered to the construction site do not have properties significantly different from the soils that were used in developing the construction specifications.

1.3.2 Clod Size

One laboratory study has shown that, for the same water content and compactive effort, permeability increased as the size of the clods increased.

The mechanism responsible has not been identified but it is assumed to be nonuniform compaction, nonuniform moisture distribution, or inadequate bonding between adjacent aggregates, leaving planes of weakness in the soil sample and areas of higher permeability.

Efforts to reduce clod size during excavation and placement of the soil for the liner would improve chances for achieving a lower permeability in several ways:

- A small clod is more likely to have a uniform water content; the nonuniform water content of larger clods will lead to differences in density and permeability after application of the same compactive effort.
- At any given water content, the material could be compacted more uniformly because the range in clod sizes would be smaller.
- If water addition were necessary, the added water could be distributed more evenly; shorter times would be required for water to move from the outside to the inside of smaller clods than of larger clods.

1.3.3 Water Content

Laboratory studies using homogenized soil samples that have been ground to pass a 2-mm sieve have demonstrated that as water content is increased several percentage points above optimum, the permeability of the soil when compacted decreases sharply. Permeability may decrease by a factor of 100 over a water content range of only 2.0 to 4.0 percent above optimum.

During construction, a uniform water content slightly higher than the optimum for maximum density is likely to result in lower permeabilities than would water contents that are optimum or less. Considerable effort may be required to control water content under field conditions but all available information indicates that a considerable return may be realized in terms of lower permeability.

1.3.4 Pilot Construction (Test Fill)

There are a number of differences between laboratory and field conditions (uniformity of material, control of water content, compactive effort, compaction equipment, etc.) that make it unlikely that permeabilities measured on laboratory-compacted samples can be achieved during construction. In addition, there are several processes and conditions that cannot be examined or anticipated through laboratory work (e.g., control of desiccation cracking, bonding of lifts, degree of compaction on sidewalls). Pilot construction (test fill) provides an opportunity to verify that the materials, equipment, and personnel that will be used to construct the liner can produce a liner that performs according to the design. Specific factors that can be examined/tested during construction of a test fill to increase the probability of achieving minimum permeability in the actual clay liner are:

- Preparation and compaction of foundation material to the required bearing strength
- Methods of controlling uniformity of material, clod size, and water content
- Types of equipment and estimates of operational conditions (e.g., number of passes) required to achieve design density, compactive effort, and permeability
- Lift thickness and placement procedures necessary to achieve uniformity of density throughout a lift and the absence of apparent boundary effects between lifts or between placements in the same lift
- Procedures for protecting against desiccation cracking or other site- and season-specific failure mechanisms for the finished liner or intermediate lifts
- Measuring the permeability on the test fill in the field and collecting samples of field-compacted soil for laboratory testing
- Test procedures for controlling the quality of construction
- Ability of different types of soil to meet permeability requirements in the field
- Skill and competence of the construction team.

1.4 SUMMARY

Clay liners are a widely used technology for management of hazardous and nonhazardous wastes. Field data on the performance of clay liners or on the effect of various construction procedures on the permeability of liners are limited. Laboratory data show that low permeabilities (less than 10^{-7} cm/s) can be achieved by compacting soils. The few case studies that have been conducted suggest that permeabilities measured on laboratory-compacted samples or on small-diameter intact samples are poor predictors of performance of the actual liner. A large body of information exists that suggests which aspects of construction practice, if changed, would be most likely to improve liner performance and increase the certainty that clay liners will exhibit uniform permeabilities with values approaching those measured in the laboratory.

1.5 REFERENCE

Haxo, H. E. et al. 1983. Lining of Waste Impoundment and Disposal Facilities. SW-870, U.S. Environmental Protection Agency, Cincinnati, Ohio.

CHAPTER 2

CLAY SOIL

Clay liners are composed of layers of cohesive soil, engineered and compacted to form a barrier to liquid migration. From an engineering standpoint, soil has been defined as unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks (ASTM, 1985) or all materials in the surface layer of the Earth's crust that are loose enough to be moved by a spade or shovel (Winterkorn and Fang, 1975).

Soil may be viewed as a three-phase system composed of solids, liquids, and gases. The solid phase is composed of inorganic and organic particles of varying shapes and sizes. The liquid phase is usually an aqueous electrolyte solution. The gaseous phase is basically air with variations in composition resulting from biological activity and chemical processes in the soil. Soils are normally characterized by the size and composition of their solid (particulate) components, with air- and water-filled voids considered together as porosity. However, the relative amounts of air and water (usually expressed as the degree of saturation) also influence soil behavior.

The term clay can be defined in several ways. Clay can refer to all soil particles less than a given size, usually 2 µm (Mitchell, 1976). In soils, this particle size range is composed of clay minerals and other components. Clay minerals give a clay soil its plastic and cohesive properties. Other components of the clay size fraction include nonclay minerals (e.g., marl and chalk), amorphous material, and organic material.

Geotechnical engineers use the term clay to refer to soils that contain enough clay size particles to affect their behavior (Holtz and Kovacs, 1981). Because of its emphasis on the engineering properties of clay as a liner material, this document uses the term clay to refer to clay soil, with clay mineral being used when referring to mineralogy and clay size being used when referring to particle size.

This chapter discusses the characteristics of clay soils and the clay minerals that are important soil constituents. It is intended to give the reader a brief overview of these materials, to present some basic definitions, and to discuss the properties of clay soils and clay minerals that influence the performance of clay liners. The formation and occurrence of low-permeability, clay-rich soils are also discussed. For a more thorough discussion of clay mineralogy or of the geotechnical behavior of soil, the reader is referred to Grim (1962, 1968), Mitchell (1976), and Perloff and Baron (1976).

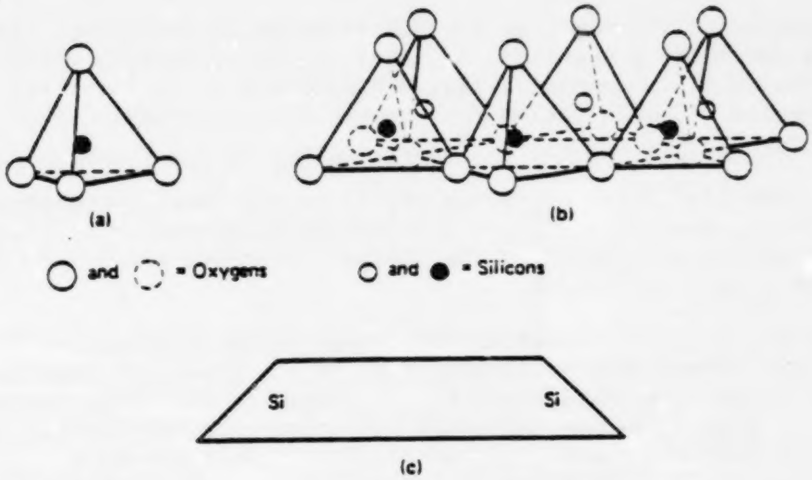
2.1 CLAY MINERALS

Clay minerals are hydrous silicates, largely of aluminum, magnesium, and iron, that, on heating, lose adsorbed and constitutional water and yield refractory material at high temperatures. Plasticity is another characteristic of clay minerals and is largely due to an affinity of the clay surface for water, resulting from a net negative charge on the surface of a clay particle that causes it to adsorb water and other polar fluids. This net negative surface charge results from defects in the clay mineral crystal structure and from surface chemical reactions, as described below. Because of their electrochemical surface activity and high surface area (resulting from their small size and lamellar shape), clay minerals can profoundly affect a soil's engineering behavior, even when present in small quantities. Other soil minerals act predominantly physically to affect a soil's behavior. As the clay content of a soil increases, the influence of the clay fraction on its behavior also increases. The strong influence of clay minerals on soil behavior can be illustrated by the addition of bentonite to a granular soil. Bentonite is a clay material composed largely of the clay mineral sodium montmorillonite. An addition of only 2 to 3 weight percent bentonite can reduce a soil's permeability when it is compacted by 2 to 3 orders of magnitude (Kozicki and Heenan, 1983). The influence of clay minerals on a soil's behavior increases with increasing clay content to a range of 33 to 50 percent, at which point the nonclay size material is essentially floating in a clay matrix and has little effect on the engineering behavior of the soil (Mitchell, 1976; Holtz and Kovacs, 1981).

2.1.1 Clay Mineral Structure

Most clay minerals have a sheet-like layered crystalline structure and thus fall into the phyllosilicate mineral family. Exceptions are the clay minerals attapulgite, palygorsite, and sepiolite, which have structures composed of double chains of silica tetrahedra. These minerals are not common in soils and are not discussed further in this document.

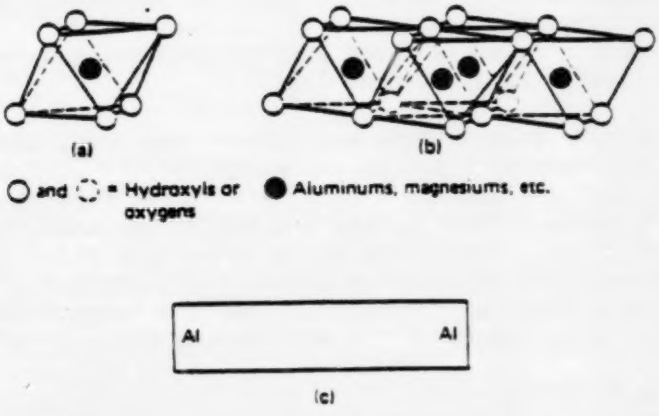
Clay mineral sheet structures consist of two different layer types, one composed of tetrahedral units and the other of octahedral units, that are arranged in different sequences to form the different clay minerals. The tetrahedral unit is composed of silica tetrahedra in which four oxygens surround a silicon atom in tetrahedral coordination (Figure 2-1). The octahedral sheet, which is made up of cations octahedrally coordinated with oxygen (Figure 2-2), occurs in two forms. If the cation is trivalent, only two-thirds of the possible spaces in a layer are filled and the structure is dioctahedral. The most commonly occurring dioctahedral sheet in clay minerals is the gibbsite sheet, in which the cations are aluminum. If the cation in the octahedral sheet is divalent, all of the available cationic spaces are filled and the structure is termed trioctahedral. The most commonly occurring trioctahedral sheet in clay minerals is the brucite sheet, in which the cations are magnesium.



(a) Silica tetrahedron.
(b) Silica tetrahedral sheet.
(c) Schematic of silica sheet. See Table 2-1.

Source: Lambe, 1958; Grim, 1968

Figure 2-1. Clay mineral tetrahedral sheet structure.



(a) Octahedron.
(b) Octahedral sheet.
(c) Schematic of gibbsite octahedral sheet. See Table 2-1.

Source: Lambe, 1958; Grim, 1963

Figure 2-2. Clay mineral octahedral sheet structure.

Isomorphous substitution, or the substitution of different, similar-size cations for those present in the ideal crystal structure without a change in structure, is common in clay minerals and is an important factor in their behavior. Common cation replacement in clay minerals includes aluminum (Al^{+3}) for silicon (Si^{+4}), magnesium (Mg^{+2}) for aluminum (Al^{+3}), and ferrous iron (Fe^{+2}) for magnesium (Mg^{+2}) in the ideal tetrahedral and octahedral sheets described above. Isomorphous substitution in clay minerals results in a charge deficiency in the crystal structure and a net negative charge on the mineral's surface.

The variety of cation substitutions, both in and between the crystalline sheets, and the intergrowth of layers of different character results in the diversity of actual clay minerals. However, despite the large number of clay minerals, only a few are important soil-forming constituents. Table 2-1, excerpted from Mitchell (1976), summarizes important chemical and physical characteristics of the clay minerals that commonly occur in soils.

2.1.2 Clay Mineral Groups

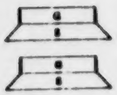
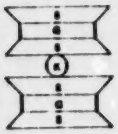
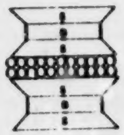
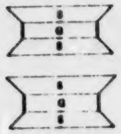
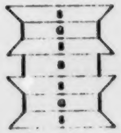
This section describes the basic structural makeup of common clay mineral groups. Although each clay mineral has a definite "ideal" structure, many naturally occurring clays are complex and do not fit the ideal formulas described herein. Mixed-layer clays can occur, with crystals containing structural units of more than one clay mineral group. In addition, soils composed of a single clay mineral or clay mineral group are relatively rare; multimineral soils are more commonly encountered in most areas. In a study of 137 soils across the United States, more than one clay mineral occurred in about 70 percent of them (Lambe and Martin, 1953-1957). Therefore, it is not possible to predict soil behavior accurately by assuming that only one clay mineral predominates through soil material. For more information on clay minerals and their complex natures, the reader is referred to Grim (1968) and Van Olphen (1963).

All clay minerals, except those with chain structure, may be roughly categorized into four groups based on the height of the unit cell, the composition of the sheets, and the kind of intersheet bonding that forms the layers of unit cells. These groups are kaolinite, smectite, illite (or mica-like), and chlorite. This grouping is convenient since members of the same group have comparable engineering behavior (Mitchell, 1976). The following subsections describe these groups and the mineral characteristics that are important determinants of the engineering behavior of clay.

2.1.2.1 Kaolinite Minerals--

The basic structural unit (unit cell) of the kaolinite group is a 1:1 arrangement of a silica tetrahedral sheet and an alumina (gibbsite) octahedral sheet (Figure 2-3). In the tetrahedral sheet, the tips of the silica tetrahedra all point toward the center of the unit. The oxygen atoms at the tips of the tetrahedra are common with one of the planes of oxygens in the octahedral sheet and compose two-thirds of the octahedral oxygens. The remaining positions in this plane are occupied by hydroxyls that are located directly below each hexagonal hole in the network formed

TABLE 2-1. CLAY MINERAL CHARACTERISTICS[a]

Type	Subgroup and schematic[b]	Mineral	Ideal formula/unit cell[c]	Cations octahedral/tetrahedral
1:1	Kaolinite	Kaolinite	$(OH)_8\ Si_4Al_4O_{10}$	Al_4/Si_4
		Halloysite (dehydrated) (hydrated)	$(OH)_8\ Si_4Al_4O_{10}$ $(OH)_8\ Si_4O_{10}\cdot 4H_2O$	Al_4/Si_4
2:1	Illite			
		Illite	$(K,H_2O)_2(Si)_8(Al,Mg,Fe)_{4,6}\ O_{20}(OH)_4$	$(Al,Mg,Fe)_{4-6}/(Al,Si)_8$
		Vermiculite	$(OH)_4(Mg.Ca)_x(Si_{8-x.}Al_x)(Mg.Fe)_6\ O_{20}\cdot YH_2O$ $x = 1\text{-}1.4,\ y = 8$	$(Mg,Fe)_6/(Si,Al)_8$
	Smectite			
		Montmorillonite	$(OH)_4Si_8(Al_{3.34}\underset{\underset{Na.66}{\uparrow}}{Mg}_{.66})O_{20}\cdot nH_{20}$[d]	$Al_{3.34}Mg_{.66}/Si_8$
2:1:1	Chlorite			
		Chlorite	$(OH)_4(SiAl)_8(Mg.Fe)_6\ O_{20}$(2:1 layer) $(MgAl)_6(OH)_{12}$ (interlayer)	$(MgFe)_6$ (2:1 layer)/$(Si,Al)_8$ $(Mg,Al)_6$ interlayer

See notes at end of table.

(continued)

TABLE 2-1 (continued)

Mineral	Isomorphous substitution	Interlayer bond	Basal Spacing	CEC[e] (meq/100g)	Specific surface (m^2/g)	Liquid limit (%)	Plastic limit (%)	Shrinkage limit (%)	Activity $\left(\frac{\text{Plasticity index}}{\% < 2\mu m}\right)$
Kaolinite	Little	0-OH Hydrogen-strong	7.2Å	3-15	10-20	30-110	25-40	25-29	0.5
Halloysite (dehydrated)	Little	0-OH	7.2Å	5-10		35-55	30-45		
(hydrated)		Hydrogen-strong	10.1Å	5-40	35-70	50-70	47-60		0.1-0.5
Illite	Si always replaced by some Al. Balanced by K between layers	K ions-strong	10Å	10-40	65-100	60-120	35-60	15-17	0.5-1
Vermiculite	Al for Si; net charge of 1 to 1.4/unit cell	Weak	10.5-14Å	100-150	40-80 primary 870 secondary				
Montmorillonite	Mg for Al; net charge always = 0.66-/unit cell	0-0; Very weak expanding lattice	9.6Å-complete separation	80-150	50-120 primary 700-840 secondary	100-900	50-100	8.5-15	1-7
Chlorite	Al for Si (2:1/layer) Al for Mg (interlayer)	0-OH Hydrogen-strong	14Å	10-40		44-47	36-40		

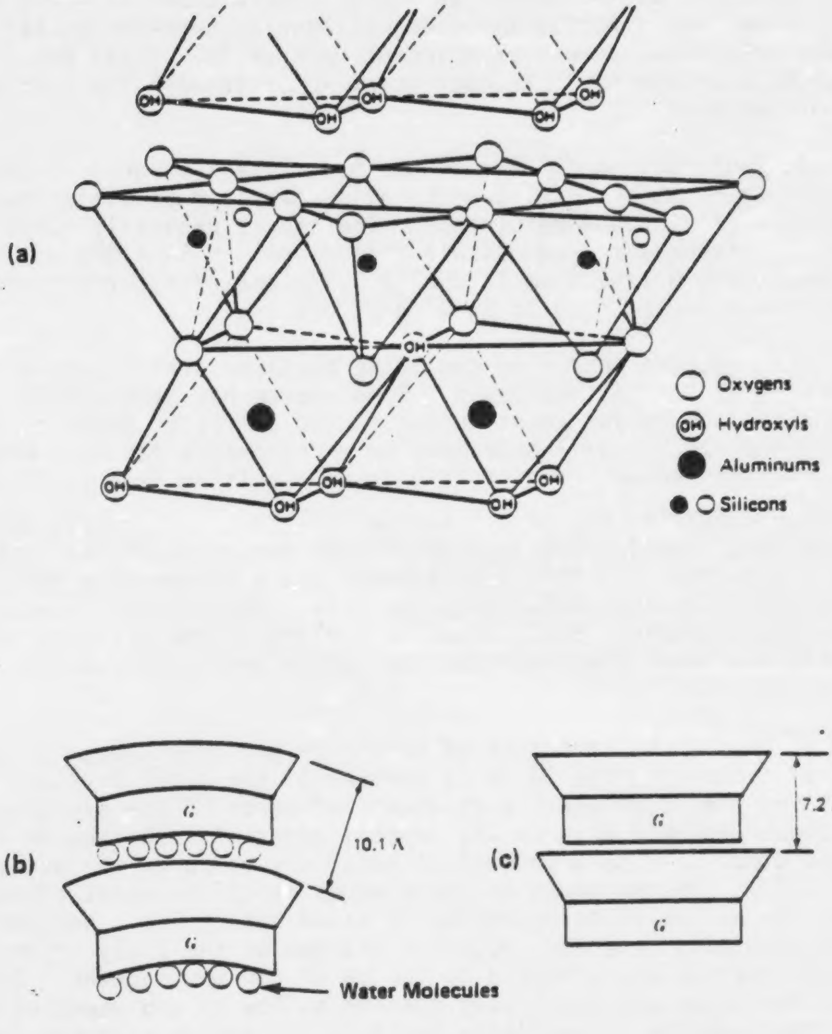
[a] After Mitchell (1976).

[b] S indicates silica tetrahedral sheet
G indicates gibbsite octahedral sheet
B indicates brucite octahedral sheet.
K indicates potassium ions
0 indicates water layer.

[c] Two formula units required per unit cell.

[d] Arrow indicates source of charge deficiency. Equivalent Na listed as balancing cation.

[e] Cation exchange capacity.



(a) Diagram of kaolinite structure.
(b) Hydrated halloysite.
(c) Kaolinite or dehydrated halloysite.

G = Gibbsite sheet.

Source: Grim, 1963; Mitchell, 1976

Figure 2-3. Kaolinite group minerals.

by the bases of the silica tetrahedral (Figure 2-3). The kaolinite basal spacing is 7.2 Å.

Minerals in the kaolinite group are composed of stacks of the 1:1 structural units (unit cells) described above. These unit cells are held together by van der Waals forces and hydrogen bonding between the tetrahedral sheets and the octahedral sheets of adjacent unit cells. These bonds are strong enough to preclude the introduction of water between the unit cells and thus any interlayer swelling.

2.1.2.1.1 Kaolinite--Kaolinite is the most common mineral in this group and consists of stacks of 1:1 unit cells comprised of silica tetrahedral and gibbsite (Al) octahedral sheets. The stacks generally range from 0.05 to 2 μm in thickness and can attain thicknesses up to 4,000 μm; the stacks can range from 0.1 to 4 μm laterally. The specific surface area of kaolinite is on the order of 10 to 20 m^2/g of dry clay.

A small net negative charge on kaolinite particles results in a cation exchange capacity of 3 to 15 meq/100 g. This charge has been attributed to a small amount of isomorphous substitution in the silica or gibbsite sheets, replacement of exposed hydroxyl hydrogens by exchangeable cations, broken bonds around particle edges, or diffuse charges resulting from the large size and surface accessibility of O^{-2} and OH^- molecules (Mitchell, 1976; Winterkorn and Fang, 1975). The stacked crystal structure of kaolinite results in a blocky form for this clay mineral and a larger size and lower surface-to-volume ratio than other clay minerals. This low surface area, combined with the relatively small negative surface charge, results in kaolinite being the least electrochemically active and least plastic clay mineral.

Because of the blocky structure of kaolinite particles, crystal edges of this mineral group comprise 10 to 20 percent of the total crystal area (Theng, 1974) and therefore exert a stronger influence on the electrochemical behavior of these minerals than do the crystal edges of smectites or illites (crystal edges comprise 2 to 3 percent of total crystal area for montmorillonite; Theng, 1974). Broken bonds on these edges result in unsatisfied valences that can be satisfied by cation or anion adsorption. However, unlike the structurally generated negative charges on the platy crystal surfaces, these charges are affected by the pH of the environment. Evidence suggests that the edges are positively charged at low pH and negatively charged at high pH. This results in kaolinite having a low cation exchange capacity at low pH and higher cation exchange capacity at high pH (see Section 2.3.1). Kaolinite has a higher anion exchange capacity than most clay minerals. This may result from the presence of replaceable hydroxyl ions on the outside of structural sheets. Kaolinite thus has the ability to fix certain negative ions (Deer et al., 1966).

Compared with other clay minerals, kaolinite has a lower affinity for water, has a lower dispersivity (see Section 2.3), and does not achieve as low a permeability upon compaction. On the other hand, because it is not as electrochemically active, its behavior may be less affected by chemicals than other clay minerals. Thus, a kaolinitic clay liner may have a higher permeability than liners composed of other clays, but the permeability of a kaolinitic clay liner may not be as sensitive to changes in moisture content or to chemical attack.

2.1.2.1.2 Halloysite--Halloysite is another kaolinite group mineral that is a common soil constituent in some areas. This mineral occurs in two forms: a nonhydrated type with a structural composition similar to kaolinite and a hydrated form with a single layer of water interposed between unit kaolinite layers (Figure 2-3). This layer increases the basal spacing to 10.1 Å, compared with 7.2 Å for nonhydrated halloysite and kaolinite. Partially hydrated halloysite (metahalloysite) with basal spacing from 7.4 to 7.9 Å can also occur. The interlayer water molecules in hydrated halloysite are believed to be in a rather flat hexagonal network linked to each other and to adjacent halloysite layers by hydrogen bonding.

The hydrated form of halloysite occurs in cylindrical tubes of overlapping kaolinite sheets. The outside diameters of the tubes range from 0.05 to 0.20 µm, with a median value of 0.07 µm, and range in length from a fraction to several micrometers. The specific surface area of halloysite ranges from 35 to 70 m^2/g (Mitchell, 1976).

Because of the interlayer water sheet in hydrated halloysite, intercalation (introduction between the unit cells) of chemicals can occur. This also results in a slightly higher cation exchange capacity for hydrated halloysite (5 to 40 meq/100 g) than for kaolinite (3 to 15 meq/100 g). Halloysite also may be more affected by chemicals than kaolinite.

The interlayer water in halloysite is easily removed during drying, and this dehydration is irreversible. Because of this phenomenon, soil engineering tests on air-dried samples can give different results than those performed on samples at the original field moisture content. For this reason, it is especially important that laboratory tests on soils with appreciable halloysite content be carried out on samples at the original field moisture content (Holtz and Kovacs, 1981; Hilf, 1975).

2.1.2.2 Illite Minerals--

The illitic clay minerals are also known as the mica-like clay minerals because of their structural similarity to hydrous micas. Illites are composed of mica-like three-layer sandwiches with an octahedral sheet between two silica tetrahedral sheets (2:1 layers). These "sandwich" layers are in turn bound together by fixed or exchangeable cations. The specific mineral species in this group are determined by differences in octahedral sheet composition and the type of interlayer cations. Two minerals in this group common in soils are illite and vermiculite.

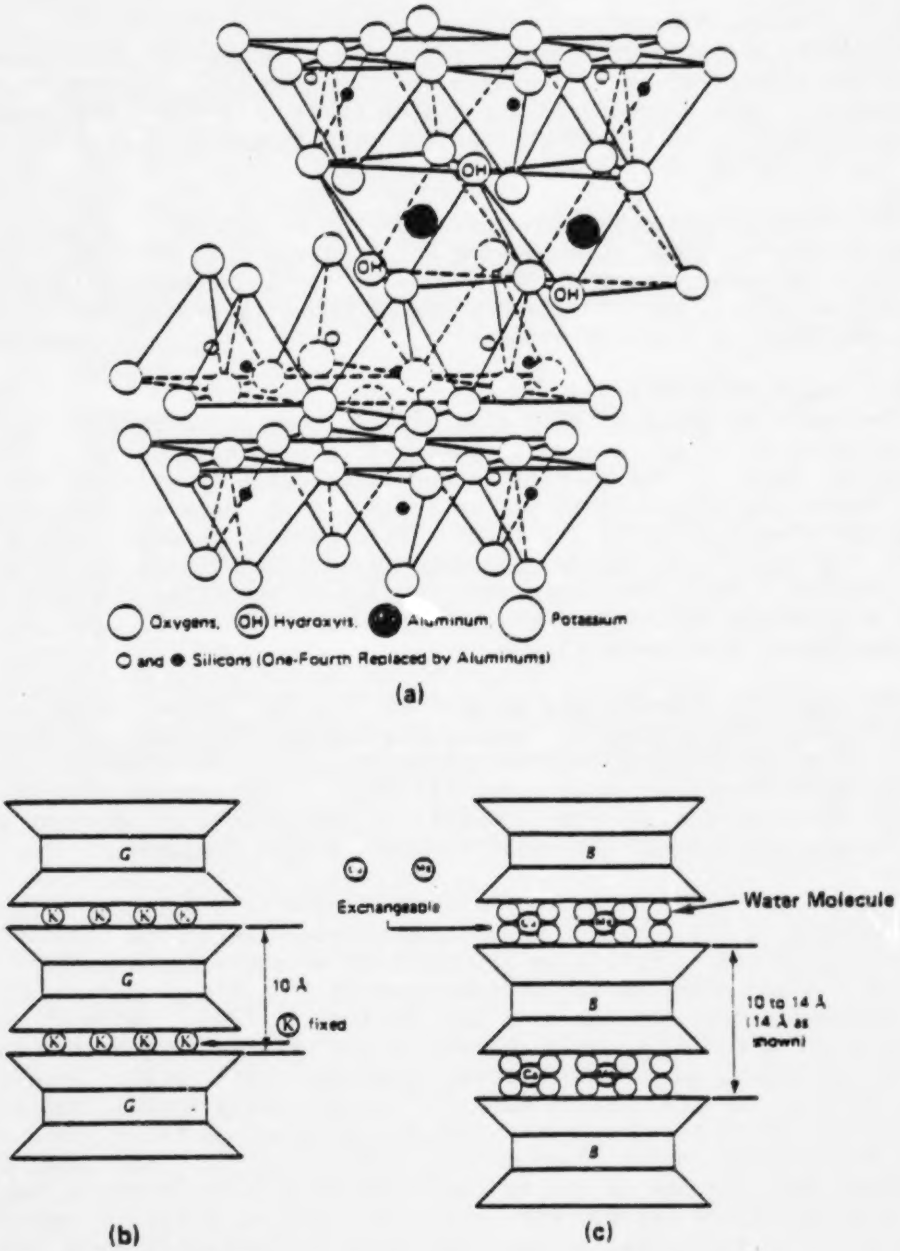
2.1.2.2.1 Illite--Illite is an important constituent of clay soils and has been described by Mitchell (1976) as "perhaps the most commonly occurring clay mineral found in soils encountered in engineering practice." Illite has almost the same crystalline structure as muscovite mica. This structure is comprised of a silica-gibbsite-silica sandwich, with the tips of the silica tetrahedra pointing toward the octahedral gibbsite sheet and the oxygens at the tips being common with the octahedral sheet (Figure 2-4). Isomorphous substitution of aluminum for silicon in the tetrahedral sheet results in a negative charge at the surface of these layers. This charge is balanced by potassium, cesium, and ammonium ions between the 2:1 layers; these ions fit tightly in the 1.32-Å-radius holes in the bases of the silica sheet and as a result are fixed in position and are not exchangeable. Illite differs from muscovite in having less isomorphous substitution in the tetrahedral sheet, a lower negative surface charge, and a lower amount of potassium between the layers. The stacking of illite layers is also more random, and illite occurs with a much smaller particle size than muscovite.

In terms of properties important to clay liner performance, illite lies between kaolinite and the smectite clay minerals. Although extensive isomorphous substitution results in a net negative charge on the clay mineral surface, the fixed potassium cations balance the charges and strongly bond adjacent 2:1 sheets together. As a result, illite has intermediate values for surface area (65 to 100 m^2/g), cation exhange capacity (10 to 40 meq/100 g), swelling index, and activity. It is also intermediate in its reaction to chemicals. Because of the strength of the interlayer potassium bonding, the basal spacing of illite remains at 10 Å when it is exposed to polar liquids (Mitchell, 1976). The potassium ions effectively prevent the intercalation of water, organic liquids, and other cations (Deer et al., 1966).

2.1.2.2.2 Vermiculite--Vermiculite is a fairly common mineral in clay soils and usually occurs with other clay minerals. Vermiculite has a 2:1 structure with a poorly organized octahedral sheet sandwiched between two silica tetrahedral sheets (Figure 2-4). The octahedral sheet contains iron and magnesium ions.

As with illite, isomorphous substitution of aluminum for silicon is extensive in the tetrahedral sheet, resulting in a net negative charge on the crystal surface. This positive charge deficiency is larger than that of the smectite minerals (see Section 2.1.2.4) and is usually balanced by interlayer layers of divalent cations and water. This larger charge deficiency results in vermiculite having the highest cation exchange capacity of all clay minerals (Deer et al., 1966). The most common interlayer cations in vermiculite are magnesium and, to a lesser extent, calcium.

The amount of water that is intercalated in vermiculite is less variable than that in smectite and usually is limited to two layers of water molecules. The interlayer spacing is therefore fairly constant for vermiculite but varies to some extent depending on the cations present between the layers. Vermiculites can absorb organic liquids between their layers but take up less than the smectite minerals (Deer et al., 1966). The primary specific surface area for vermiculite ranges from 65 to 100 m^2/g. This is within the range reported for montmorillonite and, as with montmorillonite, the secondary (interlayer) surface area can reach very high values (870 m^2/g) (Mitchell, 1976).



(a) Diagram of illite structure
(b) Illite schematic.
(c) Vermiculite schematic

G = Gibbsite sheet.
B = Brucite sheet.

Source: Mitchell, 1976

Figure 2-4. Illite clay minerals.

2.1.2.3 Chlorite Minerals--

Chlorite minerals in clay soils are almost always found in association with other clay minerals. Chlorites are composed of 2:1 layers of silica tetrahedral sheets surrounding a gibbsite or brucite octahedral sheet, with another octahedral sheet between the 2:1 mica layers (Figure 2-5). Chlorites thus may be termed 2:1:1 clay minerals.

Chlorites can have isomorphous substitution and may be missing a few of the octahedral sheets between the 2:1 layers. This can result in some swelling from water uptake between the layers. Chlorites are less active than the smectites, have a cation exchange capacity similar to illite (10 to 40 meq/10 g), and may be similar to illite in engineering behavior.

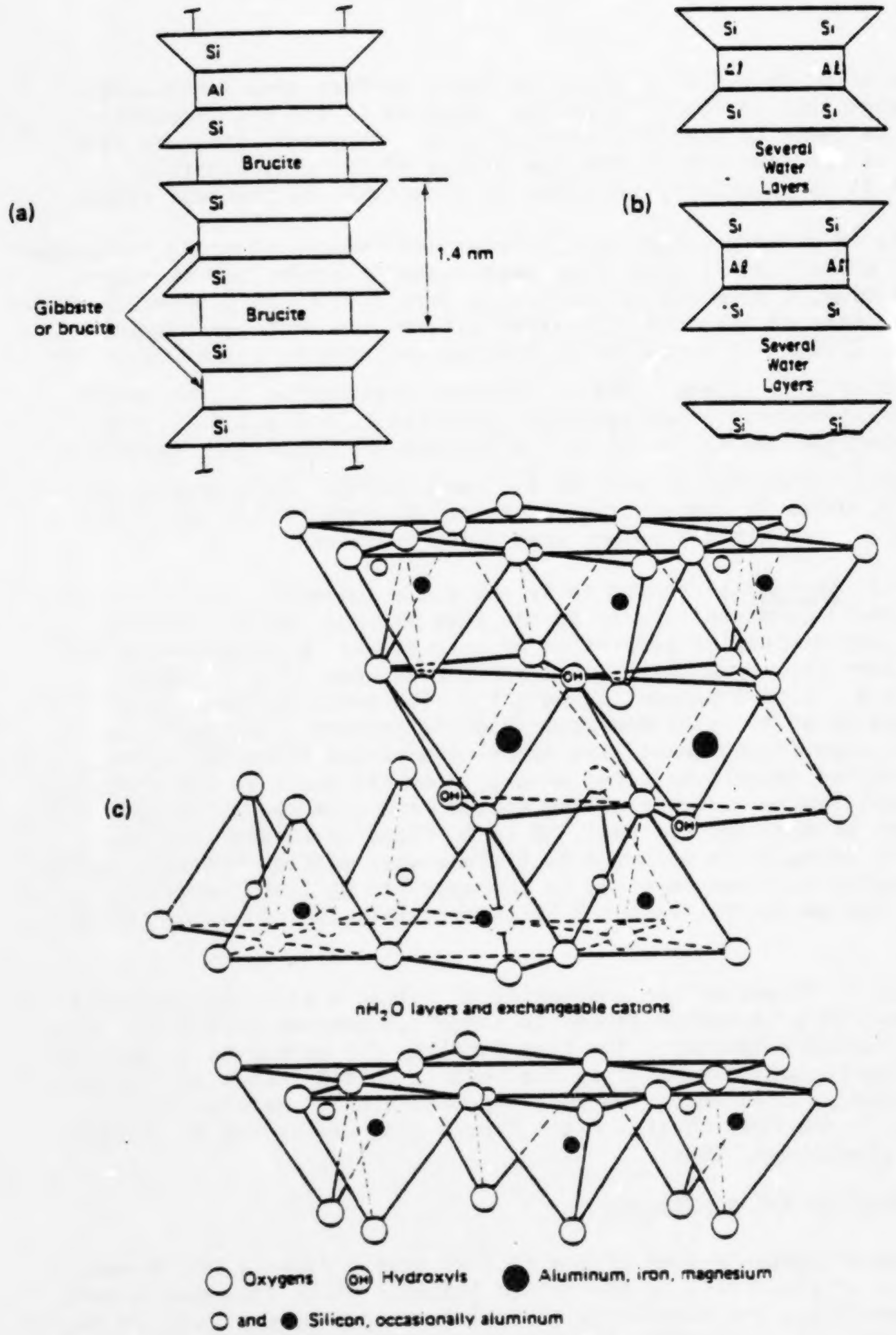
2.1.2.4 Smectite Minerals--

The smectite group of clay minerals includes 2:1 minerals whose unit cell is composed of an octahedral sheet sandwiched between two silica tetrahedral sheets. The bonding between these 2:1 layers is by van der Waals forces and cations that may be present to balance out structural charge deficiencies in the 2:1 layer. This bonding is weak, and, as a result, the layers are easily separated by adsorption of water or other polar liquids. Thus, the interlayer spacing of smectites can vary from 9.6 A to complete separation, and this results in the high swelling behavior and high activity of these clay minerals.

The smectite minerals may be divided into two groups, based on the composition of the octahedral sheet. The montmorillonites have a dioctahedral, aluminum-based (gibbsite) octahedral sheet; the saponites have a trioctahedral magnesium-based (brucite) sheet. Only montmorillonite is commonly found in soils. The saponites are relatively unimportant as soil constituents and are not discussed further in this document.

2.1.2.4.1 Montmorillonite--Montmorillonite is a 2:1 clay mineral with a dioctahedral gibbsite sheet sandwiched between two silica tetrahedral sheets (Figure 2-5). Extensive substitution of magnesium and other cations for aluminum and aluminum for silicon results in a charge deficiency of 0.5 to 1.2 (usually 0.66) on the unit cell (Mitchell, 1976). Most of the substitution in montmorillonite occurs in the octahedral gibbsite sheet, usually one magnesium for every sixth aluminum. This results in a charge on the mineral surface that is more diffuse or evenly spread than that of vermiculite, which has mostly substitution of aluminum for silicon in the outer, tetrahedral layers (Deer et al., 1966; Winterkorn and Fang, 1975). The charge deficiencies on the montmorillonite unit cells are balanced by exchangeable cations between the unit cells, and, as a result, montmorillonite exhibits a high cation exchange capacity (generally 80 to 150 meq/100 g).

The bonding forces between unit cells of montmorillonite are weak, and water and polar fluids can easily penetrate between the unit cell layers. As a result, montmorillonite particles are very small and can be dispersed to sheets of unit cell thickness (10 Å) in water (Mitchell, 1976). The specific surface area of montmorillonite is very high, with a primary



(a) Schematic diagram of chlorite.
(b) Schematic diagram of montmorillonite.
(c) Diagram of smectite structure.

Figure 2-5. Chlorite and smectite clay minerals.

surface area of 50 to 120 m^2/g and a secondary surface area (including interlayer surfaces) of 700 to 840 m^2/g. Because of its high specific surface and tendency to adsorb interlayer water, montmorillonite is very susceptible to swelling and is the most active of the clay minerals. Montmorillonite is especially sensitive to alteration by chemical attack.

The type of cations occupying the interlayer spaces strongly influences the behavior of montmorillonite. The most commonly occurring interlayer cation is calcium, a divalent cation. Like vermiculite, calcium-montmorillonites usually take up two layers of water between the unit cell layers (Deer et al., 1966). This results in limited swelling to a maximum interlayer spacing of 19 Å (Theng, 1974). However, when sodium is the interlayer cation, as occurs in the Wyoming bentonites (see 2.1.2.4.2), the amount of interlayer water is not so limited and the interlayer spacing can range from 10 Å (oven-dry) to over 50 Å (Theng, 1974). This results in high swelling, which is characteristic of sodium-montmorillonite; it can expand to 13.8 times its dry volume when fully hydrated.

2.1.2.4.2 Bentonite--Bentonite is not a clay mineral. It is a rock (or clay deposit) composed largely of the clay mineral montmorillonite. The swelling and dispersive properties of this mineral give bentonite the ability to lower the permeability of a soil, even when added in small quantities (e.g., 1 to 3 percent by weight). The swelling capacity of bentonite depends on its sodium-montmorillonite content. Low-swelling bentonite has significant quantities of calcium-montmorillonite, which, because of limited interlayer water uptake, does not swell to the extent of sodium-montmorillonite. High-swelling sodium bentonite has a liquid limit of 500 percent or more and can swell 15 to 20 times in volume. Calcium bentonite will increase in volume 0 to 5 times when wetted with water; this swelling capacity has been reported to increase 700 to 1,000 percent by treating calcium bentonites with a 0.25-percent solution of Na_2CO_3 (Fisher, 1965).

Bentonite is formed by the weathering of volcanic ash. Environmental conditions favorable to sodium bentonite formation are semiarid climate and alkaline soil and groundwater. The type locality for bentonite is Wyoming, and most sodium bentonite comes from the western United States and Canada (Hosterman, 1985). Calcium or low-swelling bentonite is mainly obtained from deposits in the Gulf Coastal Plain formed from weathering of volcanic ash deposits (Hosterman, 1984).

2.2 CLAY FORMATION AND OCCURRENCE

This section presents some of the factors that influence the formation and occurrence of clay soils in the United States. It is intended to help the reader comprehend the complexity of soil-forming processes and the degree of heterogeneity and variability that may be expected in naturally occurring soils that may be used for clay liners. It is not intended to be a complete treatise on the subject.

2.2.1 Clay Mineral Paragenesis

Clay minerals are products of weathering or hydrothermal alteration, with different minerals resulting from differences in physicochemical conditions and differences in parent material (Deer et al., 1966). Clay minerals may be found in their place of origin or may be transported and deposited in sediments. In general, acid conditions favor the removal of cations from the soil and kaolinite formation, and alkaline conditions favor the formation of other clay minerals, with the predominant type(s) of cation influencing the clay mineral species. A brief discussion of clay mineral formation follows. A more complete discussion may be found in Grim (1968), Keller (1964), and Weaver and Pollard (1973).

Kaolinite mineral deposits are formed primarily by the weathering of alkali feldspars and other silicate minerals common to sialic rock types such as granites and quartz diorite. Kaolinite occurs in residual, hydrothermal, and sedimentary deposits (Patterson and Murray, 1984). In the soil environment, acidic conditions favor kaolinite formation through the dissolution and removal of bases from the parent material. Kaolinite formation is favored where alumina is abundant and silica is scarce. Kaolinites may be formed in situ but are usually a product of weathering and transport (Deer et al., 1966). Kaolinite minerals may be found alone or in association with other clay minerals in soils. Kaolinite soils are prevalent in the southeastern United States, where they are formed from weathering, erosion, and redeposition of granitic material (Mason and Berry, 1968) and in other humid regions with intense chemical weathering and good drainage.

Illites are formed from the degradation of muscovite and from the alteration of other clay minerals. The formation of illite is favored by alkaline conditions with high concentrations of alumina and potassium (Deer et al., 1966); illite can be formed from smectites under these conditions. Illite is very common in soils and is almost always found in association with other clay minerals; its stability is responsible for its abundance.

Vermiculite in soils is formed from alteration of biotite by weathering; it is usually found in association with other clay minerals in soils. Chlorite is also commonly found in soils in association with other clay minerals; it may be formed from the alteration of other clay minerals (e.g., montmorillonite) in the presence of magnesia or by the degradation of ferro-magnesian minerals. Alkaline environments rich in iron and magnesium favor its formation.

In general, the formation of smectites is favored by alkaline conditions and the presence of calcium and magnesium. However, smectites occur in association with illite in the prairie soils of the midwest (pH 4.5 to 5). Calcium is usually the dominant exchange cation in smectites, except for deposits in the western United States, where high-sodium-montmorillonite-content (sodium bentonite) clay deposits are formed from the alteration of volcanic rocks. Montmorillonite is also found in soils formed by the weathering of basic igneous rocks; poor drainage conditions promote the formation of montmorillonite in soils because magnesium is not removed during weathering (Deer et al., 1966). High calcium and magnesium and low potassium and sodium are also favorable to montmorillonite formation, as are semiarid and arid climates (Mitchell, 1976).

2.2.2 Clay Soil Formation and Occurrence

Clay soils occur throughout the United States in a variety of depositional environments. The characteristics of a clay soil are influenced by the parent material of the soil and the soil-forming processes (e.g., weathering, deposition). Clay soil may occur as transported soil or residual soil. Transported clay soil includes soil laid down by water action (fluvial soil) and soil deposited by ice action (glacial deposits). Residual soil is soil that is chemically weathered in place from the parent bedrock (USDI, 1974). Each of these soil-forming processes results in soils with different characteristics. The processes involved in soil formation are diverse and multivariate, resulting in a wide range of chemical and physical variations.

2.2.2.1 Fluvial Soils--

Fine-grained, high-clay-content soils are found in the outer edges of torrential outwash deposits or alluvial fans (deposits laid down where an abrupt flattening of stream gradient occurs), especially in humid climates. Other alluvial (steam laid) deposits can also have high clay contents; sediments from older, slow-moving, meandering streams have a larger fraction of fine-grained components (USDI, 1974). Overbank, floodplain deposits laid down in still waters also tend to be fine-grained.

Individual strata of alluvial deposits can vary considerably, both vertically and horizontally and in clay content and permeability (Freeze and Cherry, 1979). A predominantly fine-grained floodplain or valley-fill deposit should be expected to have many lenses or strata of coarse-grained, permeable material. Complex vertical and lateral facies changes, from fine-grained overbank deposits to coarse-grained river channel deposits, are common in valley-fill and valley terrace deposits. These lateral facies changes result from changes in a river course and changes in a flow regime and reflect past and present geomorphic processes and environments. Meandering stream deposits are especially variable in this regard because of the continuous changes in river course common to this geomorphic environment. Thus, there is much heterogeneity in alluvial sediments resulting from textural variability. Hydraulic conductivities can vary more than 2 or 3 orders of magnitude in these deposits (Freeze and Cherry, 1979). Because of this heterogeneity, knowledge of fluvial processes can be very helpful when site investigations are conducted in areas of alluvial soils. Alluvial deposits are found in river valleys across the United States, in glaciated regions, and in the large intermountain basins of the Southwest (Heath, 1984).

Lacustrine deposits are deposited by the still water of lakes. They are fine-grained, finely stratified deposits with low permeability, high compressibility, and low shear strength. Lacustrine deposits have flat surfaces and are surrounded by high ground (USDI, 1974). Near the edges of lake deposits, alluvial influences can result in lenses of coarse-grained, permeable material being interstratified with the fine-grained lake sediments. As with the valley-fill deposits, these heterogeneities can result in rapid groundwater migration through the permeable layers. Lacustrine deposits are common in glaciated regions, along the Gulf coastal plain, and in the intermountain basins of the West (Freeze and Cherry, 1979).

Because they are laid down in still or slow-moving water, fine-grained, low-permeability fluvial deposits tend to have a dispersed soil fabric (see Section 2.4.1 for a discussion of fabric). However, flocculated fabrics also can result from fluvial deposition. Fluvial soils tend to be anisotropic with respect to permeability from stratification, with permeability greater in the horizontal than in the vertical direction. In one study of alluvial and lacustrine sediments (Johnson and Morris, 1962), 46 samples had a greater horizontal than vertical hydraulic conductivity, 11 samples were isotropic, and 4 samples had greater vertical conductivities. Horizontal conductivities were 2 to 10 times larger than vertical conductivities in this study.

2.2.2.2 Glacial Soils--

Glacial deposits are those that are laid down by the advances and retreats of the North American continental ice sheets and by smaller mountain glaciers in the Rocky Mountains, the Sierra Nevadas, and Alaska. Glacial soils tend to be very heterogeneous because the source material of a single soil can extend over hundreds of miles. Deposits of the glacier proper are referred to as glacial-till or morainal deposits and are heterogeneous, well-graded deposits with particle sizes ranging from clay to large boulders. They are deposited directly from glacial ice with little or no sorting by moving water. The rock fragments are generally contained in a matrix of fine silt and clay, resulting in a low-permeability soil with considerable shear strength. These are excellent qualities for landfill sites and clay liner material. For these deposits, the fine clay material can have a flocculated, random fabric because till is deposited in mass from the wasting ice. Fine-grained glaciofluvial and glaciolacustrine deposits tend to have a dispersed fabric because these deposits are fluvial in origin (see preceding discussion on fluvial soils). However, flocculated fabrics can result from fluvial deposition.

The heterogeneities common in glacial soils make a detailed site survey necessary during investigations of a landfill site or borrow pit in glacially deposited soils. Field permeability tests should be part of any site surveys in glacial soils because of the potential for fractures and stratified heterogeneities. Knowledge of glacial processes is very helpful in delineating heterogeneities during a site investigation in glacial soils.

2.2.2.3 Residual Soils--

Residual soils result from the in situ weathering of underlying bedrock and are always present in the place where they have formed. The character of a residual soil is determined by the parent bedrock and the type and degree of weathering to which it has been subjected (USDI, 1974). Residual soils are rare in glaciated regions because the action of ice removed such soils during the last glaciation, and not enough time has passed since then (10,000-12,000 years) for weathering processes to form new soils from rocks in place. Residual soils generally occur where transported soils or surficial bedrock are absent (USDI, 1974).

The mineralogy and physical grain morphology for residual soils are different from those of other soils. Residual soils are more leached than glacial or fluvial soils, with lower lime, magnesium, and potash contents and higher silica, alumina, and iron-oxide contents. Individual mineral grains in residual soils tend to be more rounded and weathered (decomposed) than in other soils. The fabric of residual soils can be flocculated, or randomly oriented, as the mineral grains are formed in place and not deposited with preferential orientation as are fluvial soils. However, weathering processes can also result in dispersed fabric, depending on the mineralogy of the soil and the geochemical environment (see Section 2.4.1.1).

Although most soils have unique compaction curves, for a residual soil, compaction curves at a given compactive effort can change, depending on the soil moisture content at the start of the test. This has been attributed to the presence of halloysite (which exhibits irreversible drying) and to particle breakdown during air drying and subsequent compaction (Hilf, 1975). Irreversible changes during drying can also occur for other soil types (see Section 3.8.3.1; Sangrey et al. 1976).

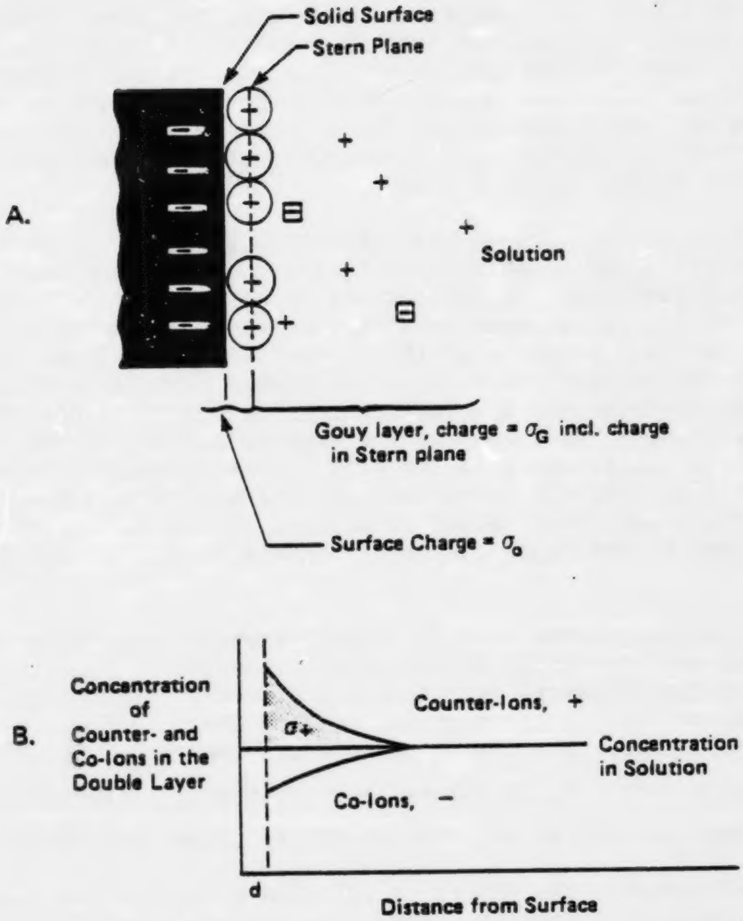
2.3 CLAY CHEMISTRY

2.3.1 Electrical Double-Layer Theory

Most clay mineral particles fall into the size range that defines colloids. Colloids are particles that are sufficiently small to allow interfacial forces to be significant in their behavior. For clay minerals in soil environments, these interfacial forces are present as a charged solid-solution interface that gives rise to an electrical or diffuse double layer.

An electrical double layer is composed of a layer of charges fixed on or within the surface of a solid and a layer of mobile ions of opposite charge, or counter ions, distributed in the liquid adjacent to the surface. These counter ions are weakly held close to the surface by electrostatic forces and can be displaced with respect to the charged, solid surface with movement of the liquid.

The theory of the diffuse double layer was originally developed by Gouy (1910) and Chapman (1913), with later modifications by Stern (1924) and others, to explain colloid behavior. Parks (1975) and Stumm and Morgan (1970) describe several models of the double layer. A simple model is illustrated in Figure 2-6. In this case, electrostatic attraction between the adsorbent (negatively charged clay surface) and adsorbate (positive ions in solution) results in a layer of cations at the surface (the Stern



Solid Surface
Stern Plane
A.
Solution
Gouy layer, charge = σ_G incl. charge in Stern plane
Surface Charge = σ_o
B.
Concentration of Counter- and Co-Ions in the Double Layer
σ_+
Counter-Ions, +
Concentration in Solution
Co-Ions, −
d
Distance from Surface

After Parks, 1975.

Figure 2-6. Electrical double layer.

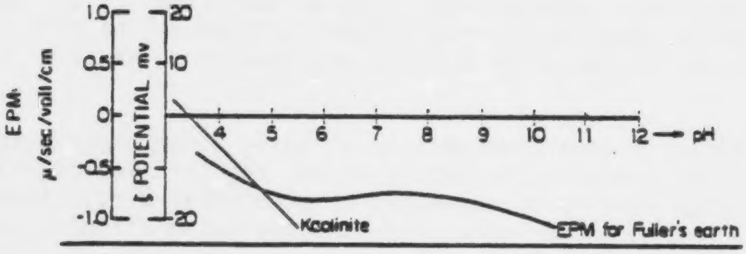
layer) and a diffuse layer of cations in solution (the Gouy layer). The cation concentration in the Gouy layer decreases with distance from the negatively charged surface. The thickness of the Stern layer is fixed by the hydrated radius of the adsorbed counter ions; the plane through the center of the Stern layer counter ions is known as the Stern plane (Figure 2-6a). The thickness of the Gouy layer is defined as the distance from the charged surface at which the cation and anion concentrations reach that of the bulk solution. From this point, for a negatively charged surface, cation concentrations increase and anion concentrations decrease as one moves toward the surface (Figure 2-6b).

The negative surface charge that generates the electric double layer around clay minerals may be attributed to two causes (Stumm and Morgan, 1970): (1) imperfections or substitutions in the crystal lattice of the clay minerals (i.e., isomorphous substitution) and (2) charge deficiencies arising from chemical reactions at the mineral surface. These surface chemical reactions include those involving broken bonds at particle edges and noncleavage surfaces and the interactions of potential-determining ions (H^+ or OH^-), with hydroxyl groups at the exposed surface of the clay particle. Isomorphous substitution is the major cause of negative surface charge for all clay minerals except perhaps the kaolinite group. Surface chemical reactions contribute about 20 percent of the surface charge for smectites and may be the major source of surface charge for kaolinites (Mitchell, 1976).

Negative surface charge from structural causes is not affected by the pH of the solution surrounding the surface in question. However, charges arising from surface chemical reactions are directly affected by solution pH. Decreasing pH lessens the negative surface charge. For a given solution and clay mineral, there is a characteristic pH, the pH_{pzc}, at which the surface charge is zero. At pH values lower than pH_{pzc}, the surface becomes positive and thus can act as an anion exchanger (Stumm and Morgan, 1970).

Figure 2-7 compares the effect of solution pH on surface charge, in terms of electrophoretic mobility (EPM), for kaolinite and fuller's earth, a clay material usually composed principally of calcium montmorillonite. One may see that kaolinite is strongly affected by solution pH, with fuller's earth being much less affected. This supports the contention that most of the surface charge for kaolinite is due to surface chemical effects. Most of the total surface charge of the 2:1 minerals (e.g., montmorillonite) results from defects in the crystal structure inside the minerals and therefore is less affected by changes in pH.

The thickness of the double layer of clay minerals results from a balance between the electrostatic attraction force holding the diffuse cation layer to the surface of the mineral and osmotic forces tending to diffuse cations away from the mineral surface. The electrostatic attraction force depends on the mineral properties (i.e., surface charge), while the osmotic forces depend upon the properties of the solution (including dissolved ions) surrounding the clay mineral.



Source: Stumm and Morgan, 1970

Figure 2-7. Effect of solution pH on clay mineral surface charge (EPM).

A more detailed discussion of these effects may be found in Mitchell (1976) and in Chapter 4 of this document, which discusses the different effects of chemicals on clay permeability. Quantitative treatment of the electrical double layer also is discussed in Chapter 4. However, it is difficult to apply the electrical double layer equations to predict the behavior of clay soils because of the complexity of the system. Clay soils are almost always composed of a variety of clay minerals, each with different double layer characteristics.

2.3.2 Cation-Exchange Capacity and Cation Affinity

The cation-exchange capacity (CEC) of a clay mineral may be defined as the excess of cations in the electrical double layer that can be exchanged for other cations in the bulk solution (i.e., the shaded area in Figure 2-6b). The three-layer (2:1) clay minerals exhibit the largest exchange capacities. Typical CEC ranges for clay minerals are presented in Table 2-1. Ranges in CEC result from variations in composition and environmental factors; a given clay mineral does not have a single CEC value (Mitchell, 1976). It should also be noted that because clay soils are rarely composed of a single clay mineral and because other nonclay substances (e.g., sand and rock flour) usually constitute a significant portion of the soil mass, the cation exchange capacities of bulk soils are lower than those of pure clay minerals.

Cations differ in their probability of being adsorbed by a colloid. There is a well-defined affinity series of cations for clay mineral surfaces. For cation-exchange equilibria dominated by electrical dipole interactions between the cations and water molecules, the following affinity series exist:

$$Cs^{+} > K^{+} > Na^{+} > Li^{+}$$

and

$$Ba^{++} > Sr^{++} > Ca^{++} > Mg^{++} > Be^{++} .$$

Thus, affinity increases with decreasing ionic radius for a given cation valence. This series does not apply to ion exchange sites that have affinities for specific cations (e.g., interlayer illite potassium sites).

As predicted by double layer theory, clay minerals have a greater affinity for bivalent cations than for monovalent cations and this selectivity decreases with increasing solution ionic strength (Stumm and Morgan, 1970). In addition, the selectivity of clay surfaces for different ions in a mixed ion system is temperature-dependent. Mass action effects (i.e., high concentrations of specific cations in solution) can override the affinity series (Mitchell, 1976).

Cation exchange affects the double layer thickness and this, in turn, can affect the permeability of clay soil through its effects on particle arrangements. Replacement of a monovalent cation by a divalent cation results in a reduction in double layer thickness, lower interparticle repulsion and dispersion, increased flocculation, and increased permeability (see Section 2.4.2). This phenomenon is discussed in Chapter 4. The properties of clay minerals with higher cation exchange capacities (e.g., montmorillonite) are more affected by cation exchange than those of clay minerals with low cation exchange capacities (e.g., kaolinite). Cation exchange capacity increases with increasing mineral surface area and surface charge.

2.3.3 Significance of the Electrical Double Layer to Clay Liners

The thickness of the electrical double layer is an extremely important determinant of the engineering properties of a clay soil. For clay soils, increasing clay mineral electrical double layer thickness results in a more dispersed fabric and lower hydraulic conductivity (see Section 2.3). Reduction in electrical double layer thickness for clay minerals can occur through desiccation, cation exchange, and interactions with certain chemicals (see Chapter 4 for further discussion of clay/chemical interactions). Reduction in double-layer thickness results in a more flocculated soil structure and increased permeability. The effect of double-layer thickness in a clay soil on its behavior also depends on the size and specific surface area of the clay particles in the soil. The behavior of smaller clay minerals, with higher specific surface area and surface charge (e.g., smectites), is more affected by changes in double-layer thickness than clay minerals with lower specific surface areas (e.g., kaolinite). Relative sizes of clay minerals are illustrated in Figure 2-8.

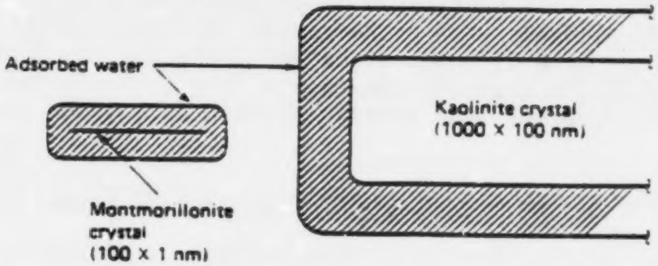
2.4 CLAY SOIL FABRIC AND HYDRAULIC CONDUCTIVITY

This section briefly discusses the fabric of fine-grained soils and the engineering property of soils most relevant to clay liners; i.e., the relationship of compaction to hydraulic conductivity (permeability). For more information on these subjects, see Grim (1962, 1968), Mitchell (1976), and Hilf (1975). A discussion of clay/chemical compatibility, including the effects of chemicals on soil hydraulic conductivity, may be found in Chapter 4 of this document.

2.4.1 Soil Porosity and Hydraulic Conductivity

Porosity and hydraulic (or fluid) conductivity are two fundamental soil properties from the standpoint of fluid transport. The pore size of a soil is the most important factor influencing the soil's hydraulic conductivity. Although a statistical relationship exists between porosity and hydraulic conductivity for relatively uniform porous media (increasing porosity increases hydraulic conductivity), it is difficult to relate quantitatively porosity and hydraulic conductivity for clay soils (Mitchell, 1976). However, this relationship can be discussed qualitatively.

(a)



(b)

Edge View	Typical Thickness (nm)	Typical Diameter (nm)	Specific Surface (km^2/kg)
Montmorillonite	3	100-1000	0.8
Illite	30	10 000	0.08
Chlorite	30	10 000	0.08
Kaolinite	50-2000	300-4000	0.015

(a) Relative sizes of hydrated kaolinite and montmorillonite crystals.
(b) Relative sizes of clay minerals.

Source: Lambe, 1958

Figure 2-8. Comparisons of clay mineral sizes and surface areas.

The physical structure* of a soil determines its porosity and its hydraulic conductivity. Two types of porosity may be delineated in soils: primary and secondary. These types of porosity are controlled by, respectively, the microstructure (fabric) and macrostructure of the soil, as described below. In addition to primary and secondary porosity, macrostructural heterogeneities (e.g., sand lenses and sand seams) also influence the hydraulic conductivity of a soil mass.

2.4.1.1 Soil Microstructure and Primary Porosity--

Primary porosity is the porosity of the soil mass or soil matrix and is controlled by the microstructure or fabric of the soil, which is influenced by the particle size distribution and the arrangement of mineral grains. Soils with low hydraulic conductivities have a significant percentage of fine material. Usually this fine-grained material is composed of clay minerals. Clay minerals generally have flat, platy particle shapes. The orientation of the clay platelets in the soil is one of the most important parameters determining effective porosity and hence hydraulic conductivity in fine-grained soils (Mitchell, 1976). Moreover, the primary cause of microscale anisotropy in fine-grained soils is the orientation of clay particles in a dispersed manner (Freeze and Cherry, 1979). Figure 2-9 illustrates some simple soil fabrics that can occur in clay soils. These are simplified pictures. Much more complicated particle arrangements usually occur, and a single soil may have many different zones of fabric.

In fine-grained, clayey soils, fabric can refer to the arrangement of individual clay platelets (Figures 2-9a and 2-9b) or to the arrangement of oriented groups of platelets (Figures 2-9c and 2-9d) and the relationship of these platelets to larger mineral grains (silt and sand). A soil's fabric helps determine its pore size. In a flocculated soil structure (Figures 2-9a and 2-9c), the clay platelets or platelet groups tend to be edge-to-face oriented. Soils with this fabric can be expected to have a larger pore matrix with hydraulic conductivity that is fairly equal in all directions. In a dispersed soil structure (Figures 2-9b and 2-9d), clay platelets or platelet groups tend to be oriented face to face, resulting in a deposit that is stratified on the microscale. A dispersed soil structure will be anisotropic with respect to hydraulic conductivity with higher matrix hydraulic conductivity in the horizontal direction (parallel to the oriented clay particles) than in the vertical direction (perpendicular to the oriented clay particles) (Mitchell, 1976). The pore size of a dispersed soil structure is smaller than that of a flocculated structure.

Hydraulic conductivities in soils with dispersed fabrics tend to be lower than the hydraulic conductivities of similar soils with flocculated structures. This phenomenon may be responsible for the difference between the measured hydraulic conductivity of a clay compacted wet of optimum and the hydraulic conductivity of the same clay compacted dry of optimum. Although equal density can be achieved in both cases, the hydraulic conduc-

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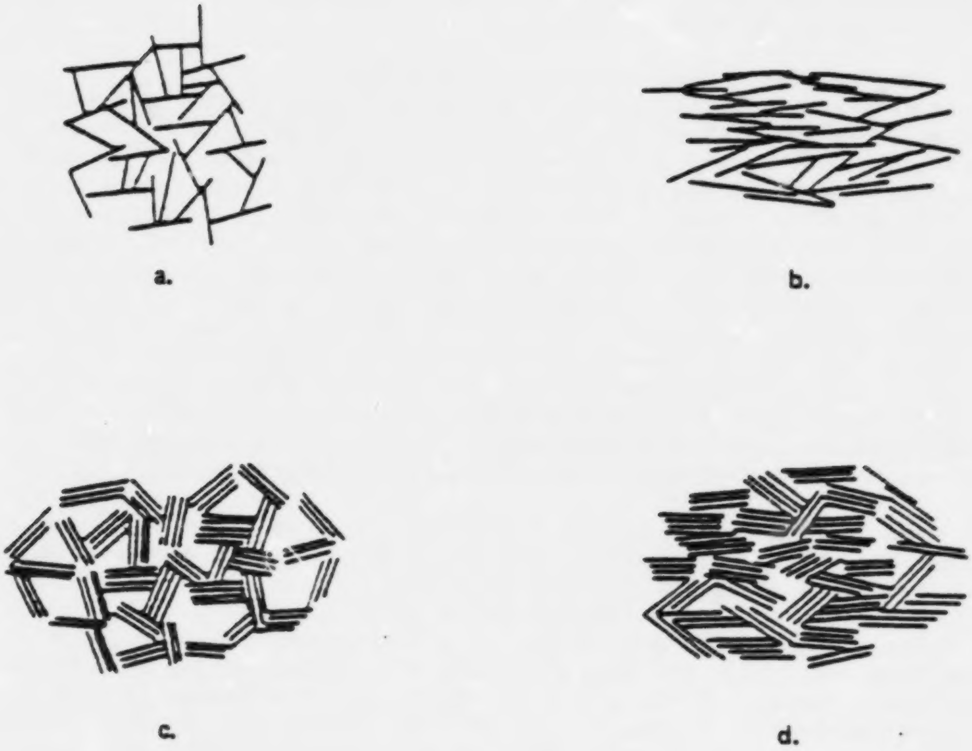
*Herein, structure refers to the physical arrangement of the soil constituents.



(a) Flocculated clay platelets.
(b) Dispersed clay platelets.
(c) Flocculated clay platelet groups.
(d) Dispersed clay platelet groups.

Source: From Holtz and Kovacs, 1981

Figure 2-9. Clay soil fabrics.

tivity of the soil compacted wet of optimum will be less (vertical direction) than that of the soil compacted dry of optimum. This difference in hydraulic conductivity can approach 2 orders of magnitude (Lambe and Whitman, 1979) and can be attributed to the increased dispersion of clay platelets and the resulting smaller pores when a soil is compacted at the higher moisture content (Daniel, 1984). Mercury porosimetry data, reported by Acar and Seals (1984) for kaolinite compacted to the same dry density, wet and dry of optimum, suggest that higher hydraulic conductivities in dry of optimum samples result from a bimodal pore size distribution; although the cumulative porosity of the two samples is the same, the dry of optimum sample has both large (~25 μm) and small ($\sim 5 \times 10^{-2}$ μm) pore sizes, whereas the wet of optimum sample has only the small pore size.

The fabric of an in situ clay soil is largely determined by soil-forming processes responsible for its formation, its depositional environment, and the action of postdepositional processes. A very important factor influencing a clay soil fabric is the electrochemical environment at the time of its deposition, including the mineralogy of the clay and the electrochemical properties of the depositional medium. These factors influence the thickness of the electrical double layer surrounding the clay particles (see Section 2.3 for a discussion of double-layer theory). Increasing the double-layer thickness increases the dispersivity of clay particles, thus decreasing their tendency to flocculate during deposition. Dispersed soil fabrics tend to occur when a clay with exchangeable sodium is deposited in water with a relatively low electrolyte level (Rogowski and Richie, 1984). In addition, dispersed, anisotropic fabric may be developed in isotropic soils from postdepositional shear or compressive forces (Mitchell, 1976). Flocculated fabric can be created in virtually all depositional environments (Holtz and Kovacs, 1981) and can result from postdepositional changes in the soil environment (e.g., cation exchange).

2.4.1.2 Soil Macrostructure and Secondary Porosity--

Secondary porosity is controlled by the macrostructure of the soil and encompasses the heterogeneities of soil deposits including joints, fissures, root and animal holes, and other defects in the soil mass (Holtz and Kovacs, 1981). Joints, fissures, and/or holes can occur in any soil as a result of stresses on the soil (i.e., earthquakes, slumping, and compression), desiccation, dissolution, or the action of vegetation or animals. In compacted engineered soils, secondary porosity can result from poor quality control of the compaction process or from desiccation of the liner after compaction.

Secondary porosity affects the hydraulic conductivity of the entire soil mass and generally overrides the effect of primary porosity. Thus, a soil that is composed of material of low matrix hydraulic conductivity can have a very high hydraulic conductivity because of secondary porosity.

Many natural clays have joints. Joints can result from postdepositional expansion and contraction from wetting and drying. Joints and fissures in preconsolidated clays result from shrinkage cracking and unloading. Cracks can also occur in clays that have a water content consistently above the shrinkage limit. These cracks have been postulated to result from syneresis--

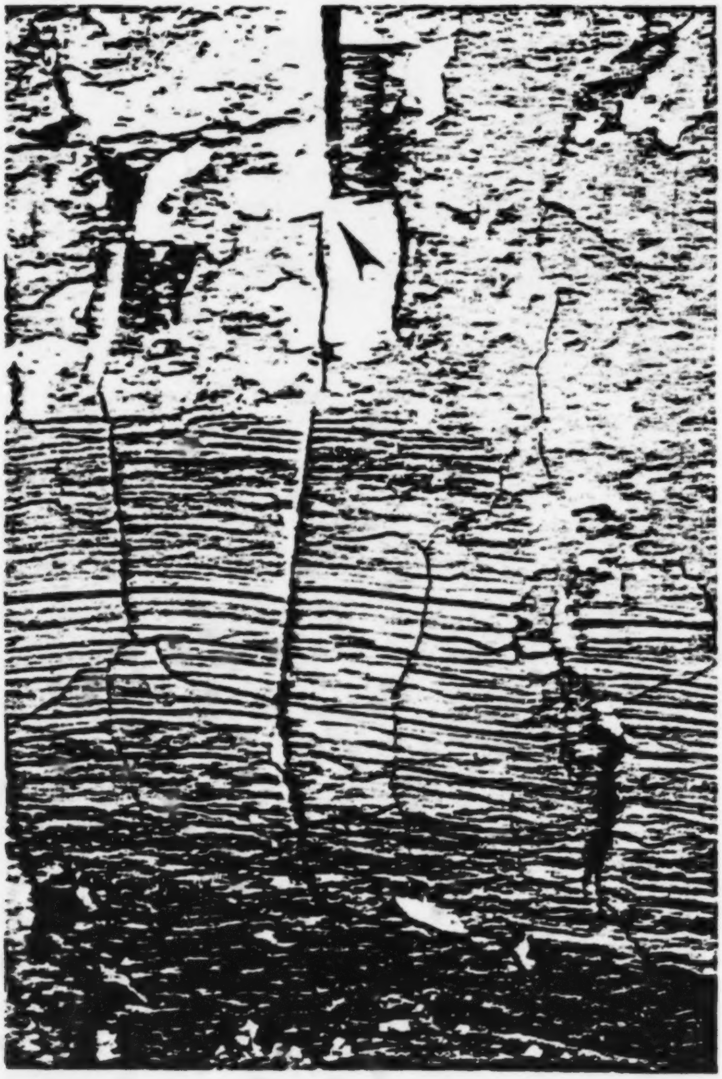
the formation of closely knit aggregates from the mutual attraction of clay particles or from the removal of potassium by weathering. Figure 2-10 shows a fracture in a glacial till with a low matrix permeability. Root systems can cause cracks by sucking up water, causing desiccation cracks to develop in the soil mass, and can leave holes in the soil mass after rotting away. Figure 2-11 shows a root cast in a till, including a fracture and a zone of higher hydraulic conductivity material left after roots have decayed.

The ability of fissures or holes to heal in a soil depends largely upon soil moisture content, soil plasticity, the size of the fissure or hole, and ambient stress. Wetter, more plastic soils have a greater healing capability (USDI, 1974). Certain organic chemicals can affect a soil's self-healing capability by reducing a soil's moisture content and plasticity.

Zones of permeable material in a soil with low hydraulic conductivity are generally referred to as lenses and seams and may be composed of silt and sand. In certain soil deposits, lenses are isolated and discontinuous and have little influence on the overall hydraulic conductivity of the soil mass and on potential contaminant migration. However, in other soils, these lenses may be continuous in one or more directions, in which case they greatly increase a soil's overall hydraulic conductivity and provide potential pathways for the rapid migration of contaminants in the groundwater. Continuous zones of permeable material are usually deposited by moving water and are common in river floodplain deposits, around the perimeter of lake deposits, and in glacial outwash deposits. Permeable zones also may be present in marine and glacial till deposits. Figure 2-12 depicts a sand seam outcropping in an otherwise low hydraulic conductivity ($K \cong 10^{-7}$ cm/s) glacial till. Leachate from a nearby landfill has migrated through this sand seam and, by evaporating, has left white salt crystals that outline the sand seam on the surface of the cut.

For residual soils, products of chemical weathering of rocks in place, the primary and secondary porosity is influenced by the degree of weathering and the composition and structure of the parent rock. Fractures and fissures in the parent rock may persist in the weathered mass (although not in highly weathered deposits), affecting the secondary porosity. Hydrothermally deposited veins of quartz in the original rock can weather into veins of sand in the resulting residual soil. In addition, for soils derived from stratified rock, the different rock strata can result in a soil with strata of different hydraulic conductivities.

Anisotropy and heterogeneity are common in all soils. Anisotropy and heterogeneity can be attributed to the existence and variation of primary and secondary porosity. On a small scale, the primary cause of anisotropy in soils is the orientation of clay minerals and its effect on primary porosity. On a larger scale, a relationship exists between heterogeneity and anisotropy (Freeze and Cherry, 1979). Heterogeneities can be attributed largely to secondary porosity and stratification in low-permeability soils. Trending heterogeneities, where transport parameters (e.g., hydraulic conductivity) change gradually in a given direction, also exist in fine-grained, low-permeability soils. Trending heterogeneities are most common in soils deposited in deltas, alluvial fans, and glacial outwash plains (Freeze and Cherry, 1979).

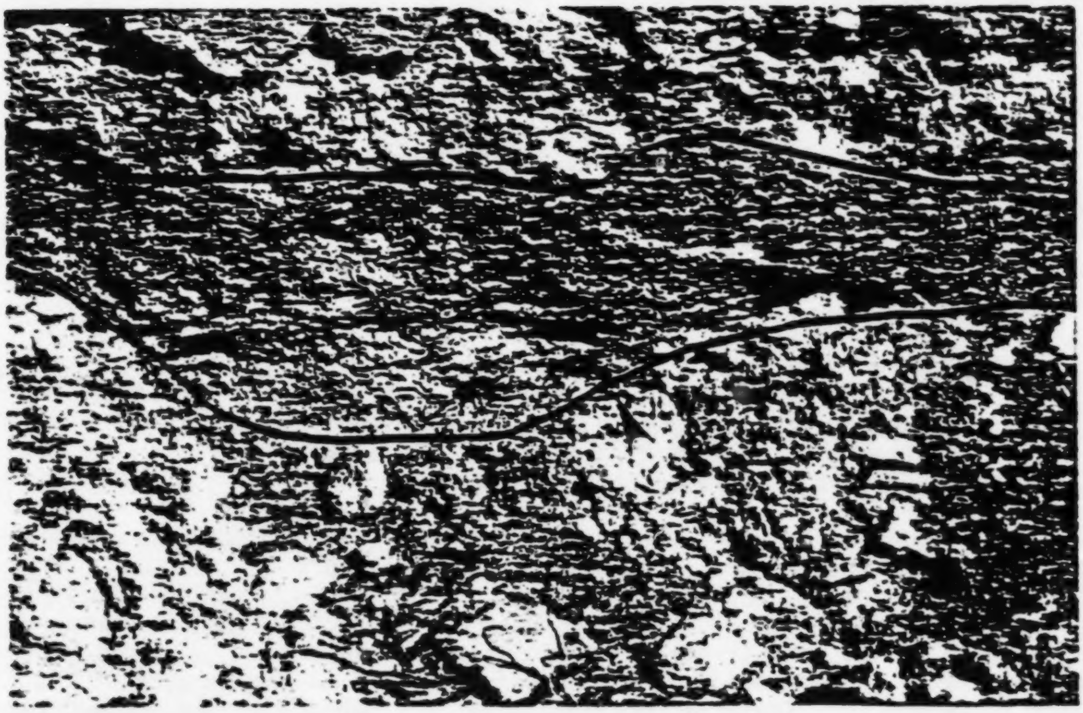


Source: Photo courtesy of Wisconsin Department of
Natural Resources

Figure 2-10. Fractures in glacial till. Note pen for scale (arrow).

Source: Photo courtesy of Wisconsin Department of
Natural Resources

Figure 2-11. Root cast in glacial till. (Root cast is the lighter colored material in the right center. Note fracture that has developed along root cast (arrow).)



Source: Photo courtesy of Wisconsin Department of Natural Resources

Figure 2-12. Permeable strata in glacial till deposit. (Permeable zone is the outlined band across the center of photo. Note white salt crystals on surface of permeable zone left by evaporating leachate flowing from a nearby landfill.)

It should be stressed that secondary porosity, if present and interconnected, can completely override the primary porosity of a soil mass, making a deposit of low-permeability matrix material highly permeable. The prevalence of heterogeneities in natural soil deposits is the primary reason that hazardous waste containment facilities should be lined regardless of the hydraulic conductivity of the in situ soil.

Heterogeneities and anisotropies in soils can lead to significant differences between field and laboratory permeability measurements (Olsen and Daniel, 1981; Griffin et al., 1985). There are at least three factors contributing to this difference:

- Most laboratory tests measure only vertical permeability, while field permeability test results usually depend on both vertical and horizontal permeability. In anisotropic soils, horizontal permeability can be higher than vertical permeability, resulting in higher field test results.

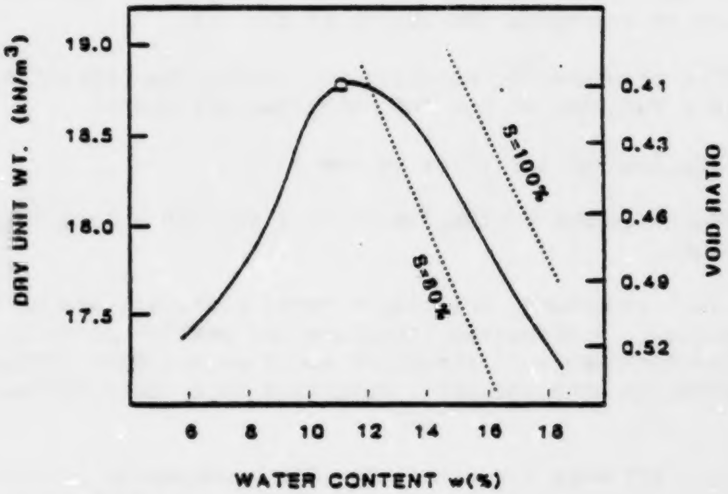
- Bias in sample selection for those cohesive enough to withstand handling and avoidance of cracked or broken samples. This leads to a predominance of measurements of matrix permeability rather than bulk permeability.

- Samples taken for laboratory permeability tests are smaller than the area covered by field tests. Field permeability tests are more likely to include heterogeneities such as fissures and zones of high-permeability material that can significantly increase soil permeability.

Because of these factors, properly performed field permeability measurements can give a more accurate estimate of soil permeability than can laboratory permeability tests that are run on "undisturbed" samples (Griffin et al., 1985; Day and Daniel, 1985). For soils with heterogeneities that affect permeability, the larger the field permeameters or the more that are used for a site investigation, the more accurate will be the measure of the soil's permeability. These factors should be considered when the protectiveness of potential hazardous waste facility sites is evaluated and during the measurement of hydraulic conductivity of compacted clay liners.

2.4.2 Soil Structure and Hydraulic Conductivity in Compacted Soils

Compaction is the application of force to a soil to reduce the percentage of air-filled voids and thus to increase its density. For cohesive soils, of which clay liners are composed, compaction with a given type and amount of compactive effort at various water contents will result in a compaction curve such as the one shown in Figure 2-13. Examination of this curve shows that, with a given compactive effort, a maximum density is achieved at a certain water content. This is the optimum water content for this type and amount of compactive effort. The "S" lines in this figure represent the computed relationship between water content and dry density at a constant degree of saturation.



Source: Lambe, 1955

Figure 2-13. Compaction curve from a standard compaction test.

Figure 2-14 illustrates that the moisture/density relationship for a given soil also depends upon the type and amount of compactive effort. This figure shows that the line of optimum moisture contents and maximum dry density for these compactive efforts approximately parallels the line of constant saturation. This figure also shows that the type of compactive effort influences the shape of the moisture content/density curve (see Chapter 3).

Much earthwork compaction is directed toward achieving structural stability, as with roadbed or foundation compaction. For these applications, strength, stability, and resistance to changes in volume are the important parameters. Density is used to control these parameters in the field. Figures 2-13 and 2-14 illustrate that the same density can be achieved either wet or dry of optimum. In many cases, foundations and roadbeds are compacted dry of optimum for greater shear strength because soil shear strength decreases with increasing moisture content (Mitchell, 1976). However, when cohesive soils are compacted to achieve lower hydraulic conductivity, both the dry density (reduction of air voids) and the fabric of the soil are important. Molding water content becomes important when attempts are made to rearrange the fabric of the clay.

The reduction of hydraulic conductivity during the compacting of a cohesive soil is a function of the following two processes:

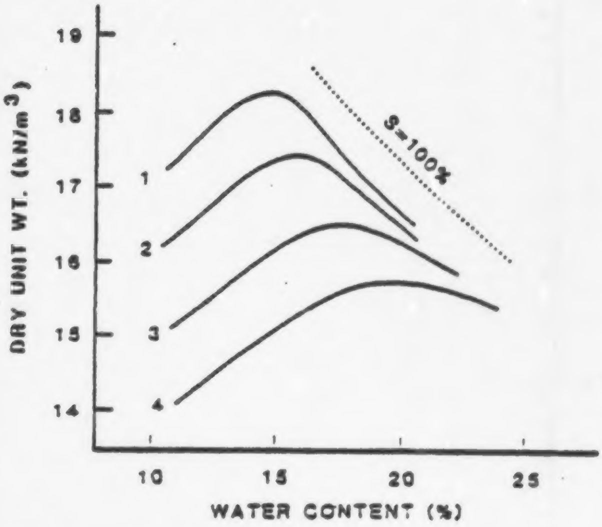
- The reduction of the voids in the soil
- The rearrangement of the fabric of a soil to a more dispersed structure.

In a dispersed soil structure, the clay mineral platelets are oriented in a face-to-face fashion. A dispersed structure has smaller pores than a more flocculated (edge-to-face) soil structure and also has more tortuous paths for liquid movement through the soil, resulting in a lower hydraulic conductivity.

Factors that influence the extent to which compaction acts on a cohesive soil fabric to produce a more dispersed structure include:

- Molding water content
- Dispersivity of the clay minerals in a soil
- Amount of compactive effort
- Type of compactive effort.

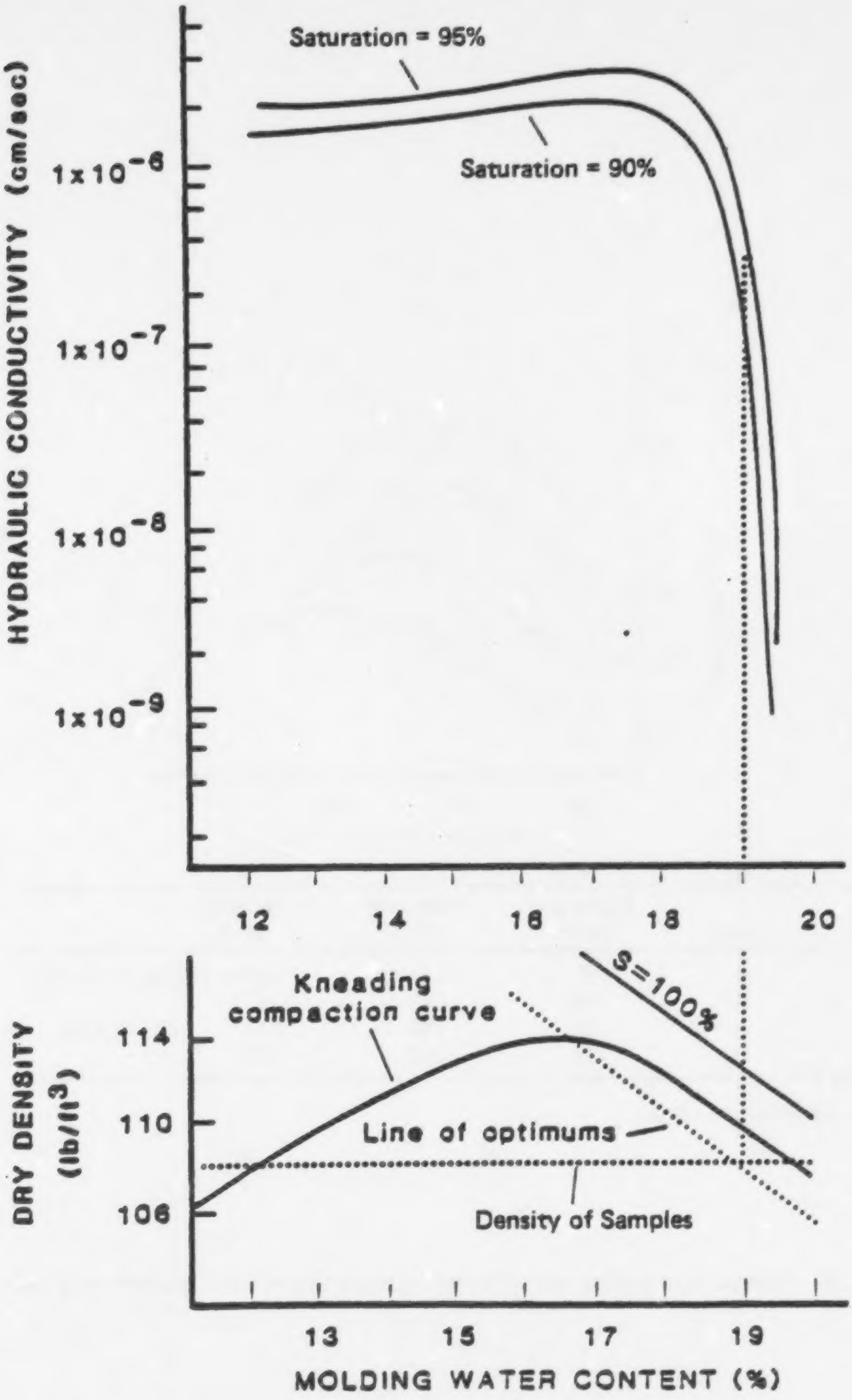
Molding water content strongly influences the ease of rearranging clay particles and particle groups under the compactive effort used (Mitchell et al., 1965; Carpenter, 1984). The profound effect of molding water content on soil hydraulic conductivity is shown in Figure 2-15. This figure illustrates that, for clay samples compacted to the same dry density



No.	Layers	Blows per Layers	Hammer Mass	Hammer Drop
1	5	55	4.54 kg	457 mm (mod. AASHO)
2	5	26	5.54	457
3	5	12	4.54	457 (std. AASHO)
4	3	25	2.50	305

Source: Lambe and Whitman, 1979

Figure 2-14. Compaction curves for different compactive efforts applied to a silty clay.



Source: Mitchell, 1976

Figure 2-15. Permeability as a function of molding water content for samples of silty clay prepared to constant density by kneading compaction.

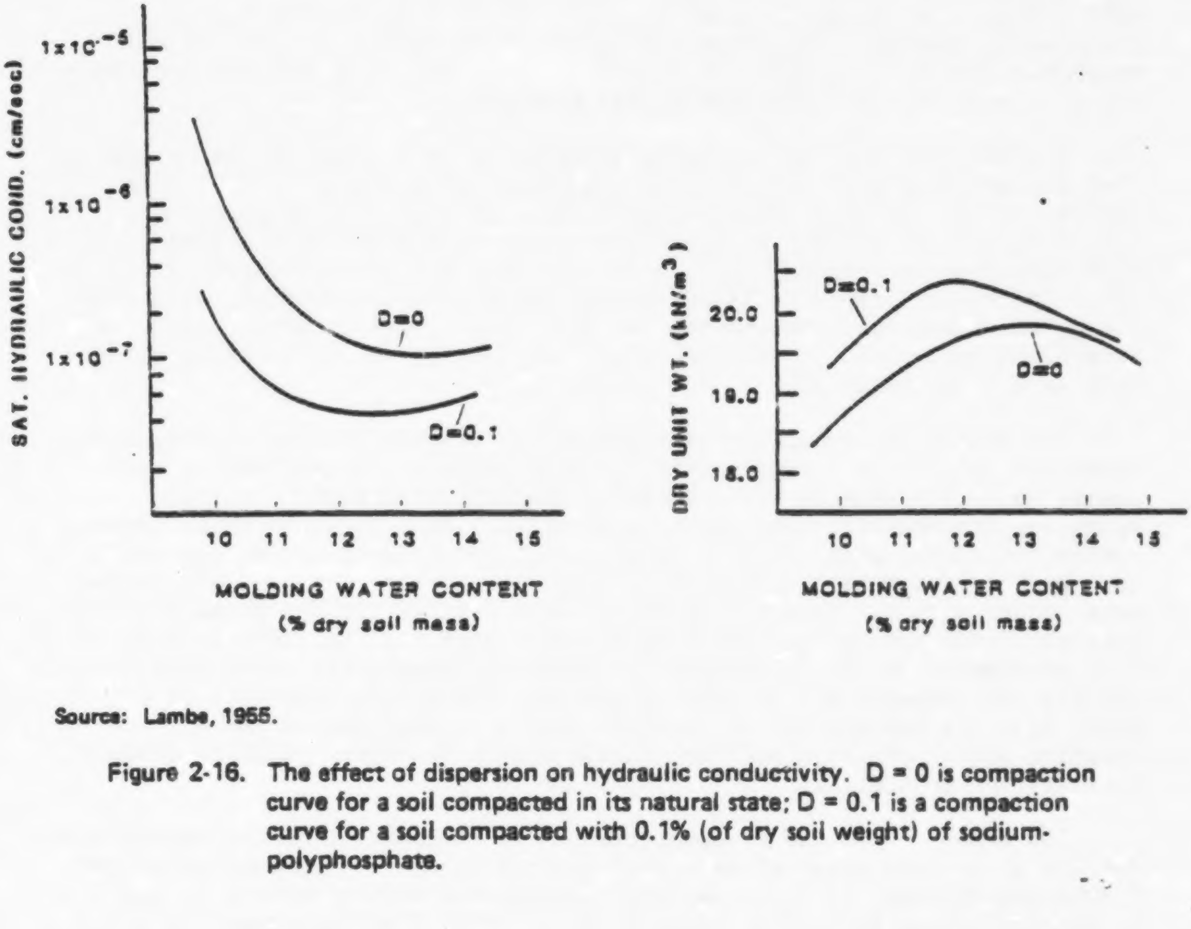
at different water contents, permeability decreases with increasing moisture content, and this decrease can be more than 3 orders of magnitude (Mitchell, 1976). Permeability has also been shown to be strongly affected by the amount and type of compactive effort used to reach the desired density (Mitchell et al., 1965). Because of these phenomena, clay liner designs should specify that clay liners be compacted wet of optimum and moisture content, density, and compactive effort measured and controlled for proper clay liner compaction. If only density is specified or measured and moisture content and compactive effort are not carefully controlled, the specified density can be achieved at too low a moisture content, resulting in a liner with a higher hydraulic conductivity than required.

Soil dispersivity is affected by the double-layer thickness of the clay minerals in the soil. Soils composed of minerals with thicker double layers have higher dispersivities. The dispersivity of clay minerals in a soil affects its hydraulic conductivity. Figure 2-16 illustrates this phenomenon, showing that adding a chemical dispersant to a soil reduces hydraulic conductivity upon compaction. (The chemical dispersant increases the double-layer thickness of the clay minerals.)

A liner soil must be carefully screened as it arrives onsite to ensure that the arriving soil type remains the same. If a change in soil type occurs, it could have a lower dispersivity. If this soil is compacted according to soil compaction parameters (density and moisture content) developed for the original soil, a more permeable liner could result. Thus, if the soil characteristics change during the construction of a clay liner, it is necessary to reestablish density/moisture content/compactive effort/hydraulic conductivity relationships for the new soil before it is used as a liner material.

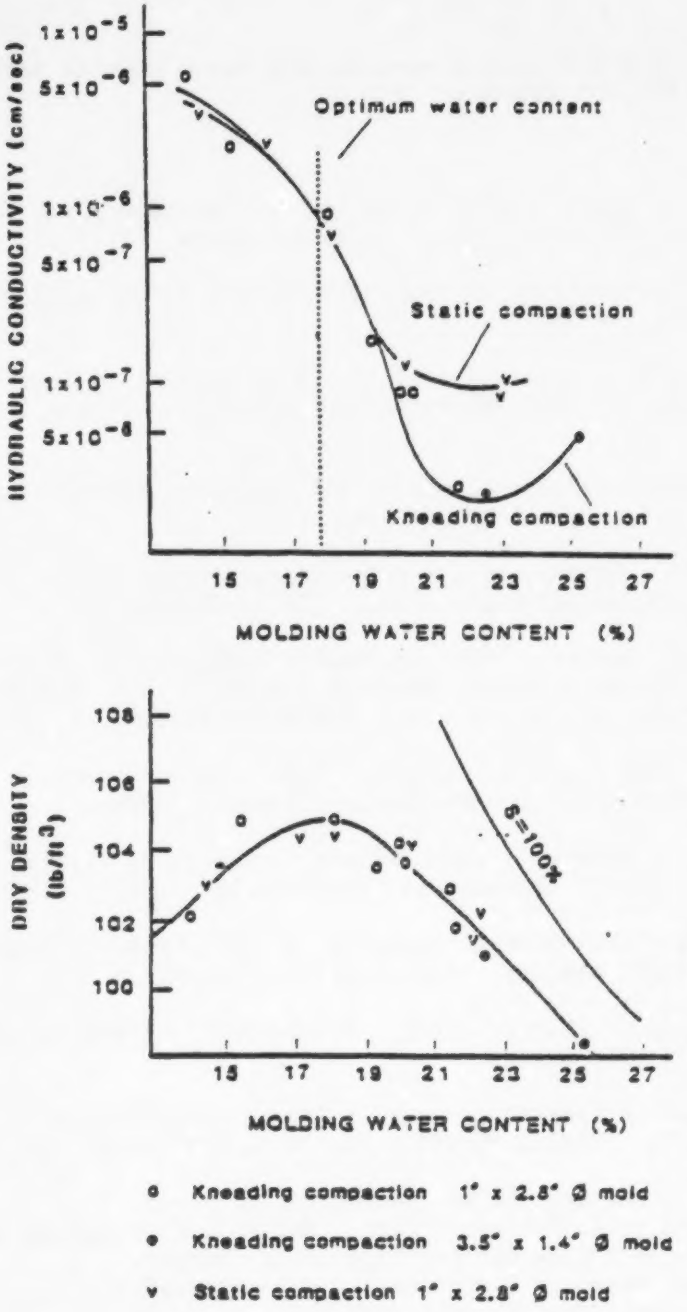
The method of compaction also affects the hydraulic conductivity of a compacted soil liner. Figure 2-17 illustrates that, for a cohesive soil compacted to the same density, kneading compaction results in a lower hydraulic conductivity than static compaction. This occurs because kneading action introduces shear strains to the soil, which tends to realign particles and results in a more dispersed soil fabric (Mitchell, 1976). This phenomenon occurs only for cohesive soils compacted wet of optimum because the clay particles can be rearranged easily only at a high moisture content. This phenomenon is the reason for a general preference for sheepsfoot rollers for compacting clay liners; although equivalent densities at a given moisture content may be obtained with a smooth-wheeled roller, the kneading action of the sheepsfoot should result in lower hydraulic conductivities for most soils.

The effect of method of compaction on soil fabric is partly responsible for the difference observed between field and laboratory compaction methods (Lambe and Whitman, 1979). Laboratory compaction methods generally result in permeabilities lower than those attained using field equipment (Dunn and Mitchell, 1984; Day and Daniel, 1985). This is because compactive efforts used in laboratory compaction tests are not the same as those produced by specific field compaction equipment. Thus it is necessary to compact a



Source: Lambe, 1955.

Figure 2-16. The effect of dispersion on hydraulic conductivity. D = 0 is compaction curve for a soil compacted in its natural state; D = 0.1 is a compaction curve for a soil compacted with 0.1% (of dry soil weight) of sodium-polyphosphate.



Source: Mitchell, 1976

Figure 2-17. Effect of method of compaction on the permeability (hydraulic conductivity) of a silty clay.

test fill in the field and conduct permeability tests on this material to verify laboratory results (see Section 5.3.3.1.1).

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CHAPTER 3

TEST METHODS AND SOIL PROPERTIES

3.1 INTRODUCTION

The engineering properties of soils are as varied as their chemical compositions, their modes of formation, and their past and present environmental settings. Consequently, when designing and constructing earthwork projects, the design or construction engineer is faced with determining whether or not a particular soil is suitable for its intended application. To aid this decisionmaking process, a wide assortment of test procedures have been developed that are used either to predict the probable engineering performance of a soil (index tests) or to measure the soil performance directly with respect to parameters of concern (e.g., permeability and shear strength).

This chapter presents and discusses the soil testing procedures that are of particular interest to designers of clay liners for hazardous waste containment facilities. Emphasis is on determining Atterberg limits and density, compaction, and permeability testing, with the bulk of the chapter devoted to the latter.

Many test procedures have been standardized by organizations like the American Society of Testing and Materials (ASTM) and are applied routinely and uniformly by all competent soil testing groups (ASTM, 1983). Permeability testing, on the other hand, has not been standardized. Different types of permeability testing equipment are in current use, and test protocols also can vary from laboratory to laboratory and even from test to test within a laboratory. The situation is further complicated by the need to measure not only the permeability to water but also, and perhaps more importantly, the permeability to leachates or other complex chemical mixtures. In addition, there is much discussion and controversy among soil experts concerning interpreting permeability test results, the relationship between field and laboratory permeability testing, and the relative merits of different types of testing equipment and test protocols.

Many soil tests, because of their routine nature and the general availability of the test protocols, are not discussed in detail this chapter. However, capsule summaries of many important tests are provided in Appendix A. These summaries have been taken directly from the EPA document Geotechnical Quality Assurance of Construction of Disposal Facilities (Spigolon and Kelly, 1984). The summaries are intended only to give the reader general information about the specific test methods. More detailed information can be obtained from the references noted for each test. A list of the summarized tests contained in Appendix A is provided in Table 3-1. Other important soil test procedures may be found in the ASTM Standards, Part 19 - Natural Building Stones; Soil and Rock (ASTM, 1983).

One of the most commonly used pieces of field test equipment is the nuclear moisture and density gauge. This device provides a rapid and reliable means of making in-place field measurements of the moisture content and density of soil. However, for accurate measurements in all test circumstances, it is necessary to understand the theoretical basis of the tests and to be mindful of certain caveats regarding the application of nuclear soil testing techniques. For these reasons, nuclear methods are described in detail in Section 3.6.2.

3.2 FUNDAMENTAL RELATIONSHIPS

The presence of fluid- and gas-filled voids in soils is an inescapable consequence of the particulate nature of the material. The multiphase nature of soils is diagramatically represented in Figure 3-1, which depicts the fundamental relationships of the weights and volumes of the basic components of a soil mass. For soil engineering purposes, the important parameters are unit weight, void ratio, porosity, and degree of saturation. The measurements that are made to compute these parameters are the weight and volume of the wet specimen, the weight of the same specimen after drying, and the volume of the solids. Discussion of the appropriate standard procedures for these measurements can be found in Laboratory Soil Testing, U.S. Department of the Army (1970); in ASTM (1983); and in Appendix A of this document.

3.2.1 Water Content

A fundamental property of any soil is its water content. This is defined as the ratio, expressed as a percentage, of the weight of water in a given soil mass to the weight of the solid particles.

3.2.2 Density

As a consequence of voids being an integral part of any soil, soil may be considered to have two "densities." One of these densities, called the specific density, is that of the solid particles. The other density, called the unit weight or bulk density, is the density of the total soil mass including water- and air-filled voids.

Specific gravity (of solids), Gs, is defined as the ratio of the density of solid particles, P_s, at a stated temperature to the density of distilled water, P_w, at 4 °C:

$$G_s = \frac{P_s}{P_w} \text{ or } G_s = \frac{W_s}{V_s \cdot \gamma_w} , \qquad (3.1)$$

TABLE 3-1. SOIL TESTS SUMMARIZED IN APPENDIX A

Method	Method number[a]
Parameter measured: water content	
Standard oven-dry	1
Standard nuclear moisture/density gauge	2
Gas burner	3
Alcohol burning	4
Calcium carbide (speedy)	5
Microwave oven	6
Infrared oven	7
Parameter measured: unit weight	
Standard laboratory volumetric	8
Standard laboratory displacement	9
Standard field sand-cone	10
Standard field rubber balloon	11
Standard field drive-cylinder	12
Standard nuclear moisture/density gauge	13
Parameter measured: specific gravity	
Standard laboratory	14
Parameter measured: grain-size distribution	
Standard sieve analysis (+200 fraction)	15
Amount of soil finer than No. 200 screen (wash) standard	16
Standard laboratory hydrometer (-200 fraction)	17
Pipette method for silt and clay fraction	18
Decantation method for silt and clay fraction	19
Parameter measured: liquid limit	
Standard multipoint	20
Standard one point	21
Parameter measured: plastic limit	
Standard laboratory	22
Parameter measured: cohesive soil consistency	
Standard unconfined compression	23
Field expedient unconfined compression	24
Hand penetrometer	25
Hand-held torvane	26
Parameter measured: water content/density/compactive effort	
25 blow standard Proctor compaction	27
25 blow modified Proctor compaction	28
Nonstandardized Proctor compaction	29
Rapid, one-point Proctor compaction	30
Rapid, two-point Proctor compaction	31
Hilf's rapid	32
Ohio Highway Department nest of curves	33
Harvard miniature compaction	34

[a] Method numbers have been assigned for the sole purpose of providing an easy way of finding the test method in the appendix. The number is meaningless outside of the context of this document.

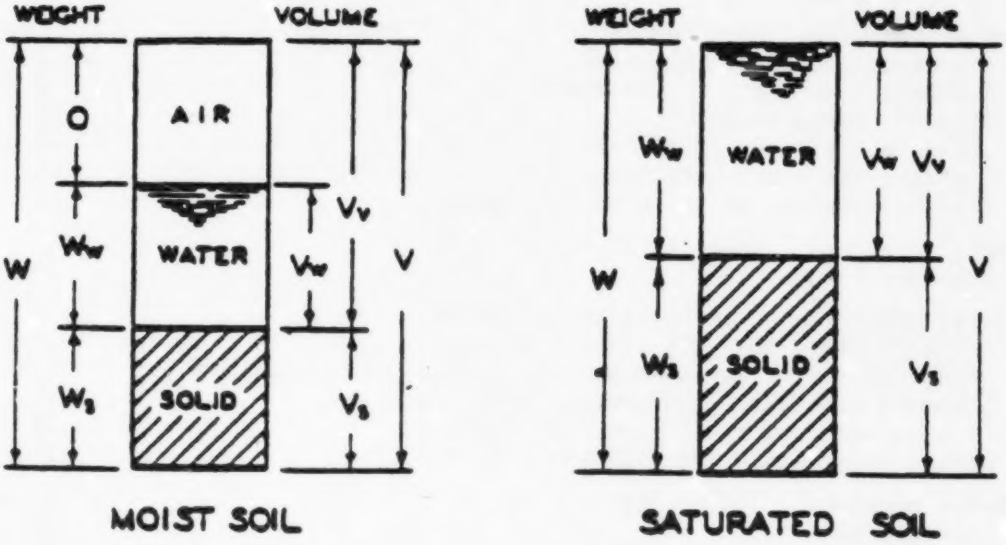


Figure 3-1. Schematic representation of soil illustrating the fundamental relationships among the solid, liquid, and air constituents.

where

γ_w = the unit weight of water (1.00 g/cm^3 or 62.43 lb/ft^3) at 4 °C.

Dry unit weight (dry density) γ_d is defined as the weight of the oven-dried soil solids per total volume of the wet soil mass:

$$\gamma d = \frac{W_s}{V} \quad . \tag{3.2}$$

Wet unit weight (wet density) γ_m is the total weight (solids and water) per unit volume of soil:

$$\gamma m = \frac{W}{V} \quad . \tag{3.3}$$

Both wet and dry density are usually expressed in pounds per cubic foot (lb/ft^3).

3.2.3 Void Ratio

Void ratio, e, is the ratio of the volume of the voids to the volume of the solids in a given soil mass:

$$e = \frac{V_v}{V_s} = \frac{V - V_s}{V_s} \quad , \tag{3.4}$$

where

V_s = volume of solids = $\frac{W_s}{G_s\ \gamma_w}$.

3.2.4 Porosity

Total porosity, n, is the ratio, usually expressed as a percentage, of the total volume of the voids of a given soil mass to the total volume of the soil mass:

$$n = \frac{V_v}{V} \times 100 = \frac{V - V_s}{V} \times 100 \quad . \tag{3.5}$$

Effective porosity is defined as that fraction of the total volume through which flow can occur. If all of the pores and void space contributed to flow, the effective porosity would simply equal the volume of voids divided by the total volume. However, some pores in soil are isolated or dead-end pores that do not contribute to flow. Water that is tightly adsorbed to the wall of a fluid-conducting pore also decreases the flow area and effective porosity. The total porosity typically ranges from 0.2 to 0.5 for clays; however, the effective porosity in clay is on the order of 0.01 to 0.1 (Zimmie et al., 1981; Daniel, 1981).

3.2.5 Degree of Saturation

Degree of saturation, S, is the ratio (expressed as a percentage) of the volume of the water in a soil mass to the total volume of the voids.

$$S = \frac{V_w}{V_v} \quad . \tag{3.6}$$

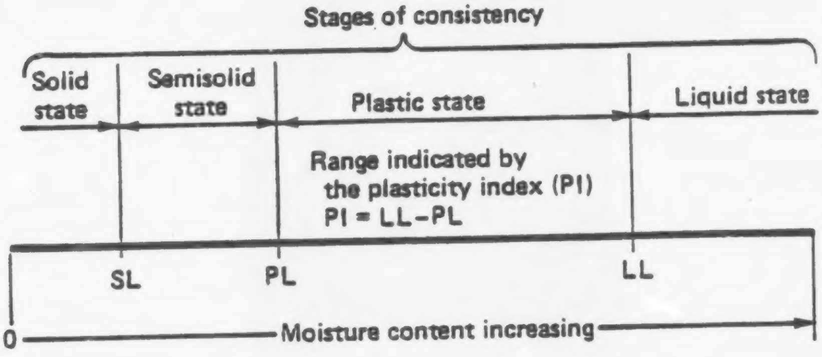
3.3 ATTERBERG LIMITS

Depending upon the water content, the consistency of a cohesive soil may range from that of a viscous liquid to that of a very hard solid. This range of consistencies may be arbitrarily divided into the four stages illustrated in Figure 3-2, with the divisions between these states referred to as the limits. These limits are called the Atterberg limits after the Swedish soil scientist, A. Atterberg, who in 1911 developed a series of tests for determining the plasticity of cohesive soils.

To understand the significance of the Atterberg limits, assume that a very wet fine-grained soil is slowly dried. In the very wet condition, the soil will act like a viscous liquid. As it dries, a reduction in volume occurs that is very nearly proportional to the water loss. When the water content is reduced to the value of the liquid limit, the soil consistency becomes plastic (i.e., the soil mass can be shaped or deformed without cracking). The liquid limit is defined as the water content, expressed as a percentage of the weight of the oven-dried soil mass, at which the soil shows a small but definite shear strength as it is dried. Conversely, with increasing moisture, it is the water content at which the soil mass first starts to become liquid under the influence of a standardized series of shocks as delivered in the liquid limit test (U.S. Department of the Army, 1970; U.S. Department of Interior, 1974).

The liquid limit test is performed with the device illustrated in Figure 3-3. This piece of equipment is designed so that the cup holding the grooved soil sample is tipped up with a cam and allowed to drop onto the hard surface below it, thereby supplying a "blow" to the soil sample. Figure 3-4 is an illustration of the groove being cut into a soil sample. The liquid limit is defined as the water content at which one-half inch of the groove is closed after 25 blows (U.S. Department of Interior, 1974).

In practice, while it is hard to adjust the soil moisture so that the groove closes at exactly 25 blows, the liquid limit can be determined by



Source: U.S. Dept. of Interior, 1974

Figure 3-2. Consistency limits of cohesive soils.

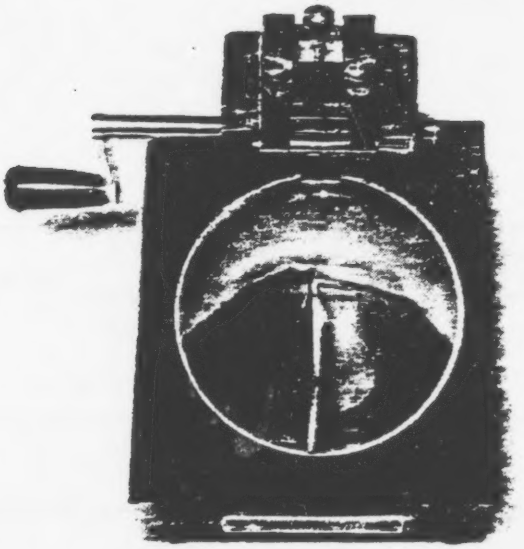


Figure 3-3. Device for determining the liquid limits of a cohesive soil. The dish contains a grooved sample.

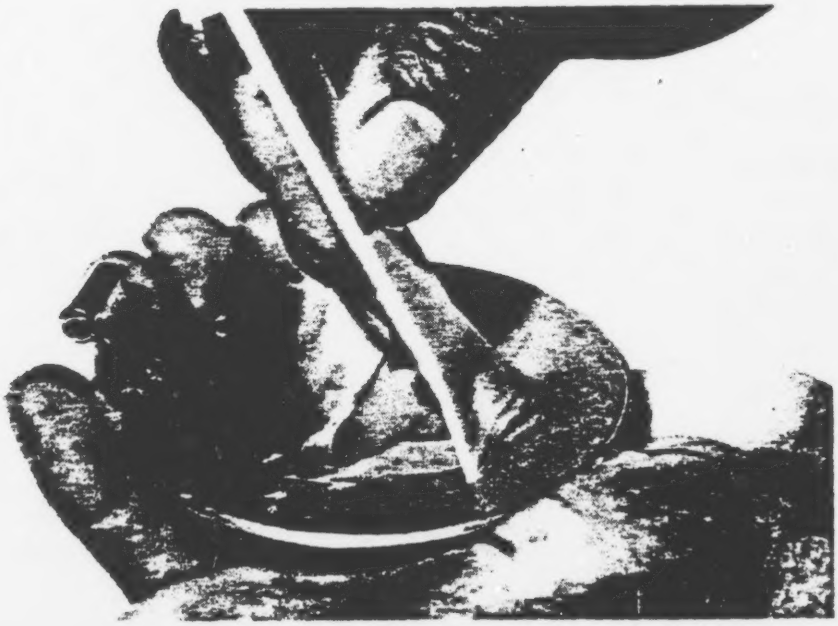


Figure 3-4. Clay sample being grooved for liquid limit test.

plotting the water contents at which closure occurs with other blow counts against the log of the number of blows needed to achieve groove closure. This procedure produces a straight line known as the flow line from which the water content corresponding to groove closure at 25 blows can be found easily. This method is ASTM D 423-66 (ASTM, 1983).

A one-point liquid limit test also can be performed provided that the practitioner is already familiar with the soils from which the sample is obtained. This method is ASTM D 423-66 (ASTM, 1983). The basis for the one-point procedure is the fact that the slopes of the flow lines for soils within a given soil environment are generally constant. Therefore, if the flow line slope, the water content, and the number of blows to obtain closure at that water content are known, the liquid limit can be computed from the following formula:

$$LL = W_N \quad \frac{N}{25} \quad \tan\beta \quad , \tag{3.7}$$

where

W_N = water content of sample

N = number of blows required to close the groove

$\tan\beta$ = slope of the flow line.

The one-point test is usually performed on soil samples prepared to a consistency that requires 20 to 30 blows to close the groove (U.S. Department of the Army, 1970).

As the water content is reduced below the liquid limit, the soil mass becomes stiffer and will no longer flow. The soil will continue to be plastic (deformable) until the plastic limit is reached. The plastic limit is defined as the water content, expressed as a percentage of the weight of the oven-dried soil, where the soil begins to crumble when rolled into a thread 1/8 inch in diameter (U.S. Department of the Army, 1970). The rolling and crumbling are illustrated in Figures 3-5 and 3-6. The test method for determining plastic index is ASTM D D424-59.

As stated earlier, a soil mass will decrease in volume in proportion to the amount of water removed. However, if enough water is removed, a point will be reached below which very little shrinkage will occur. The moisture content at this point, which is referred to as the shrinkage limit, divides the solid from the semisolid phases in the soil consistency continuum (Figure 3-2). Below the shrinkage limit, the soil is considered a solid. In most fine-grained plastic soils, the plastic limit will be appreciably greater than the shrinkage limit. However, for coarser grained soils, with predominantly coarse silt and fine sand sizes, the shrinkage limit will be near the plastic limit (U.S. Department of Interior, 1974).

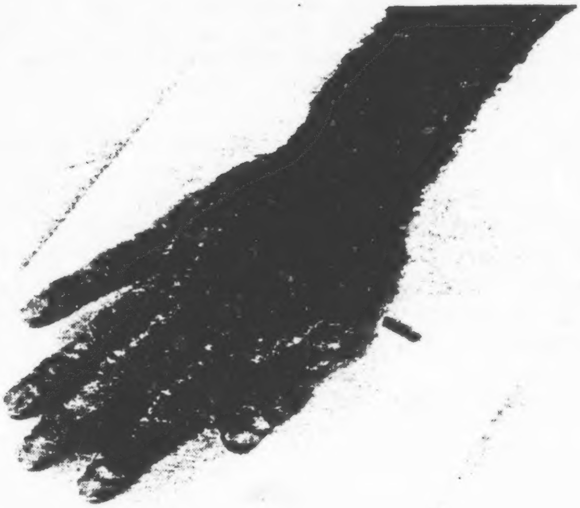


Figure 3-5. Rolling a clay sample for plastic limit test.

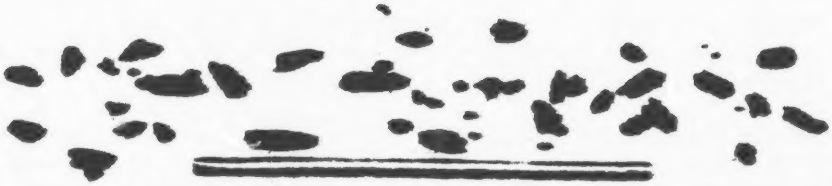


Figure 3-6. Results of rolling clay with moisture content below the plastic limit.

The test procedure for determining the shrinkage limit is ASTM D 427-61 (ASTM, 1983).

Very often the term plasticity index is encountered. It is the numerical difference between the liquid limit and the plastic limit and represents the range of moisture within which the soil remains plastic.

The plasticity index can be used to compute the activity, A, of a cohesive soil. A is the ratio of the plasticity index to the percentage of clay-size particles (smaller than 0.002 mm) and is an expression of the plasticity of the clay fraction of the soil. The mineralogy of the clay fraction is suggested by the activity, with low activity (less than 1) characteristic of kaolinite, high activity (greater than 4) characteristic of montmorillonite, and intermediate activity (1-2) characteristic of illite (Sowers and Sowers, 1970).

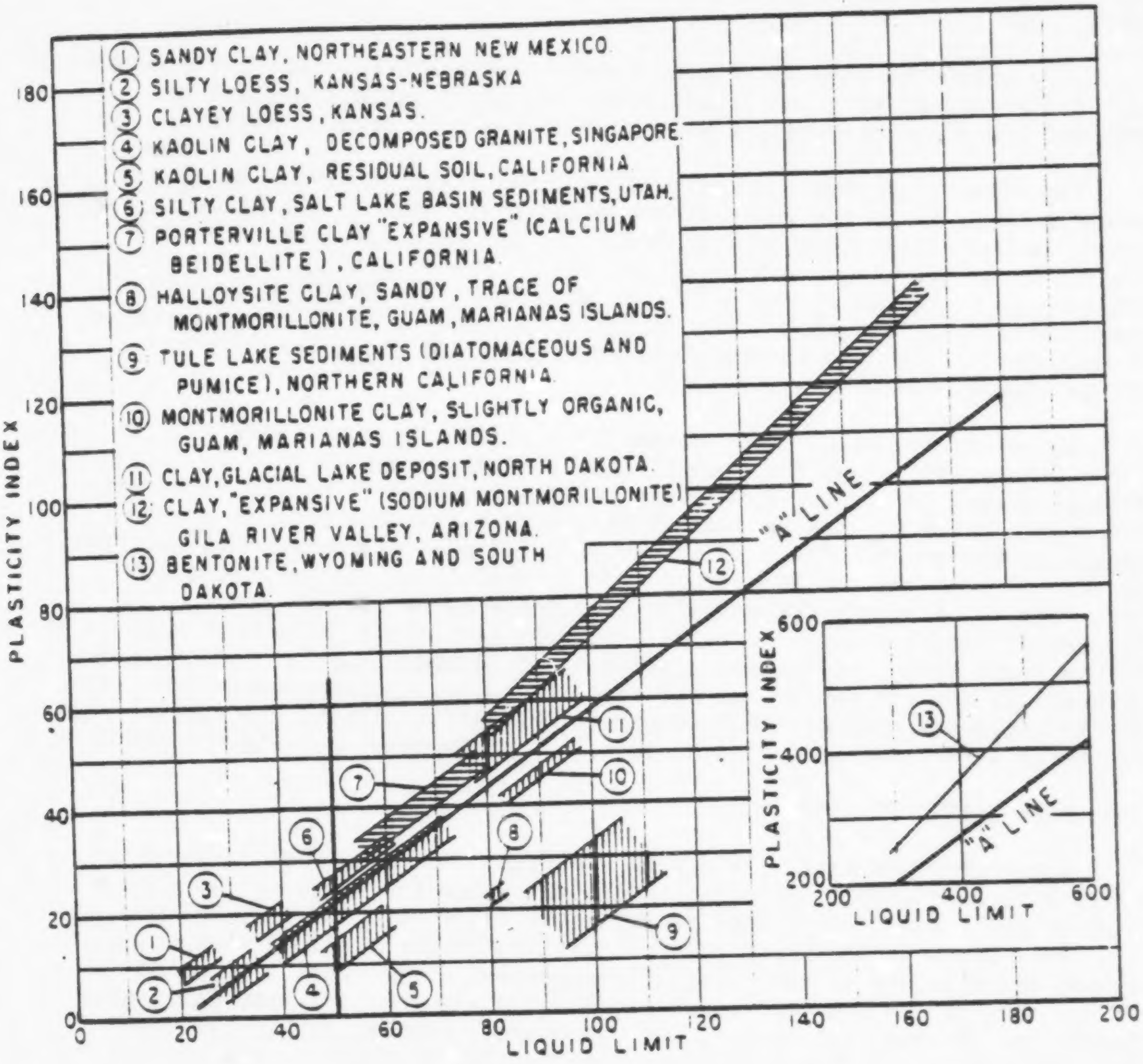
Soils may be grouped and located on a plasticity chart according to their liquid limits and their plasticity indices (Figure 3-7). The location of a soil on the chart provides a basis for estimating its engineering properties based on knowledge of other tested soils that plot in the same region of the chart. The plasticity chart is roughly divided into quadrants by the "A" line and the 50-percent liquid limit line. Soils that plot above the "A" line are classified clays; those below are not clays. The 50-percent liquid limit line is an arbitrary division between high and low liquid limit soils. In general, soils plotting above the "A" line have low permeabilities; for soils with the same liquid limit, the greater the plasticity index the greater the strength at the plastic limit; compressibility increases with increasing liquid limit (U.S. Department of Interior, 1974).

3.4 SOIL CLASSIFICATION

Based upon his early field work, Professor A. Casagrande of Harvard University developed a soil classification system for grouping soils by a plot of their plasticity index versus their liquid limit. In 1942, the U.S. Army Corps of Engineers adopted this system for airfield work. The system was modified slightly in 1947 to prevent dual classification of soils; in 1952 the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers, in consultation with Dr. Casagrande, adopted the system under the title of the Unified Soil Classification System (USCS). The USCS is descriptive, easy to use, adaptable to field or laboratory use, and takes into account the soils' engineering properties. One of its greatest advantages is that a soil can be classified readily by visual and manual examination without extensive laboratory testing. The system is based on the size distribution of the soil particles and the activity of the very fine grains (U.S. Department of the Army, 1953; U.S. Department of Interior, 1974).

3.4.1 Grain Size Analysis

Within the USCS, soils are divided into coarse-grained and fine-grained types. Coarse grains are larger than the openings in the U.S. Standard Series No. 200 sieve; fine grains can pass through the No. 200 sieve. The



Source: U.S. Department of Interior, 1974

Figure 3-7. Typical relationships between the liquid limit and the plasticity index for various soils.

particle size distribution of coarse-grained soils can be determined readily by a sieve analysis, wherein the sample is passed through a set of sieves and the material retained on each sieve is weighed (U.S. Department of the Army, 1970). Sieve sizes commonly used are listed in Table 3-2.

The size distribution of particles smaller than the No. 200 sieve can be determined by the hydrometer or by the pipette method based on Stokes' Law, which relates the terminal velocity of a sphere flowing freely through a fluid to the sphere's diameter. The relationship is expressed as:

$$V = \frac{\gamma_s = \gamma_F}{1,800\ \eta} D^2 , \tag{3.8}$$

where

V = terminal velocity of sphere, cm/s

γ_s = density of sphere, g/cm^3

γ_f = density of fluid, g/cm^3

η = viscosity of fluid, g-s/cm^2

D = diameter of sphere, mm.

The test assumes that Stokes' Law can be applied to a mass of dispersed soil particles of various shapes and sizes. The test is conducted by treating the soil sample with a chemical dispersant and then mechanically dispersing the sample in water to form a suspension. Hydrometer readings or pipette samples at a specific depth in the column are taken every several minutes, and Stokes' Law is used to compute the maximum grain size in suspension.

Methods for determining grain size distribution are summarized in Appendix A, Methods 15-19. Directions for conducting the tests can be found in U.S. Department of the Army (1970) and in ASTM Method D422-63 (ASTM, 1980).

Grain size distributions are usually displayed graphically as illustrated in Figure 3-8, which plots graded and poorly graded distributions. Well-graded soils have good representation of all particle sizes over a fairly broad size range. A poorly graded soil may have an excess or deficiency of particles within a certain size range (gap-graded or skip-graded), or all the particles occur within a fairly narrow range.

The following coefficients have been defined to determine whether a soil is well graded or poorly graded:

$$\text{Coefficient of uniformity, } C_u = \frac{D_{60}}{D_{10}} \text{ , and} \tag{3.9}$$

TABLE 3-2. U.S. STANDARD SIEVE SIZES AND THEIR CORRESPONDING OPEN DIMENSION

U.S. standard sieve number	Sieve opening (mm)
4	4.75
10	2.00
20	0.85
40	0.425
60	0.25
100	0.15
140	0.106
200	0.075

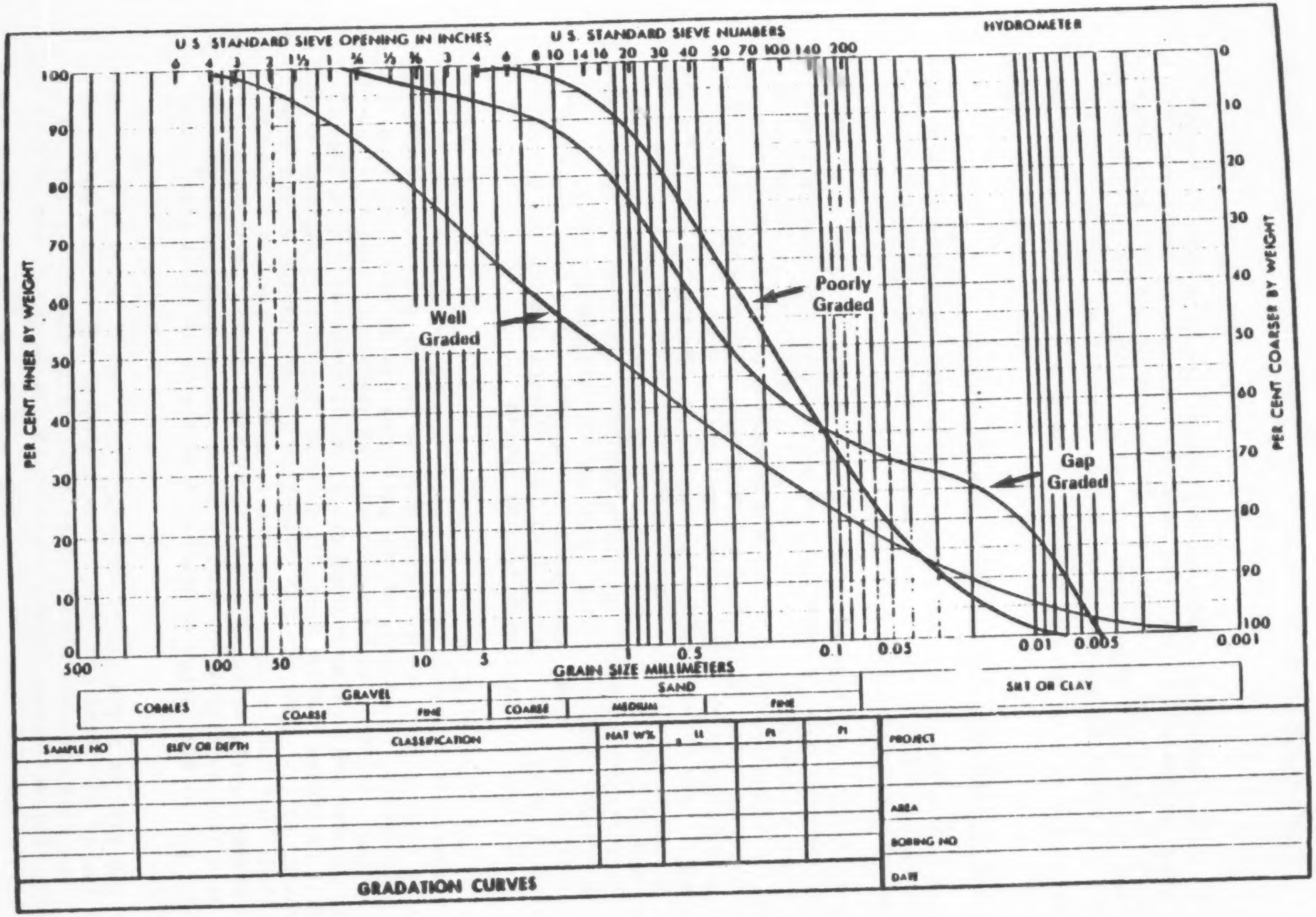


Figure 3-8. Idealized particle size distribution curves for well-graded, poorly graded, and gap-graded soils.

$$\text{Coefficient of curvature, } C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} , \qquad (3.10)$$

where "D" with a numerical subscript represents the size that is larger than a given percentage of the soil mass (as indicated by the subscript). As an example, $D_{10} = 0.5$ mm means that 10 percent of the sample by weight is smaller than 0.5 mm in diameter. To be well graded, the coefficient of curvature (C_c) must be between 1 and 3 and, in addition, the coefficient of uniformity (C_u) must be greater than 4 for gravels and greater than 6 for sands.

3.4.2 The Unified Soil Classification System (USCS)

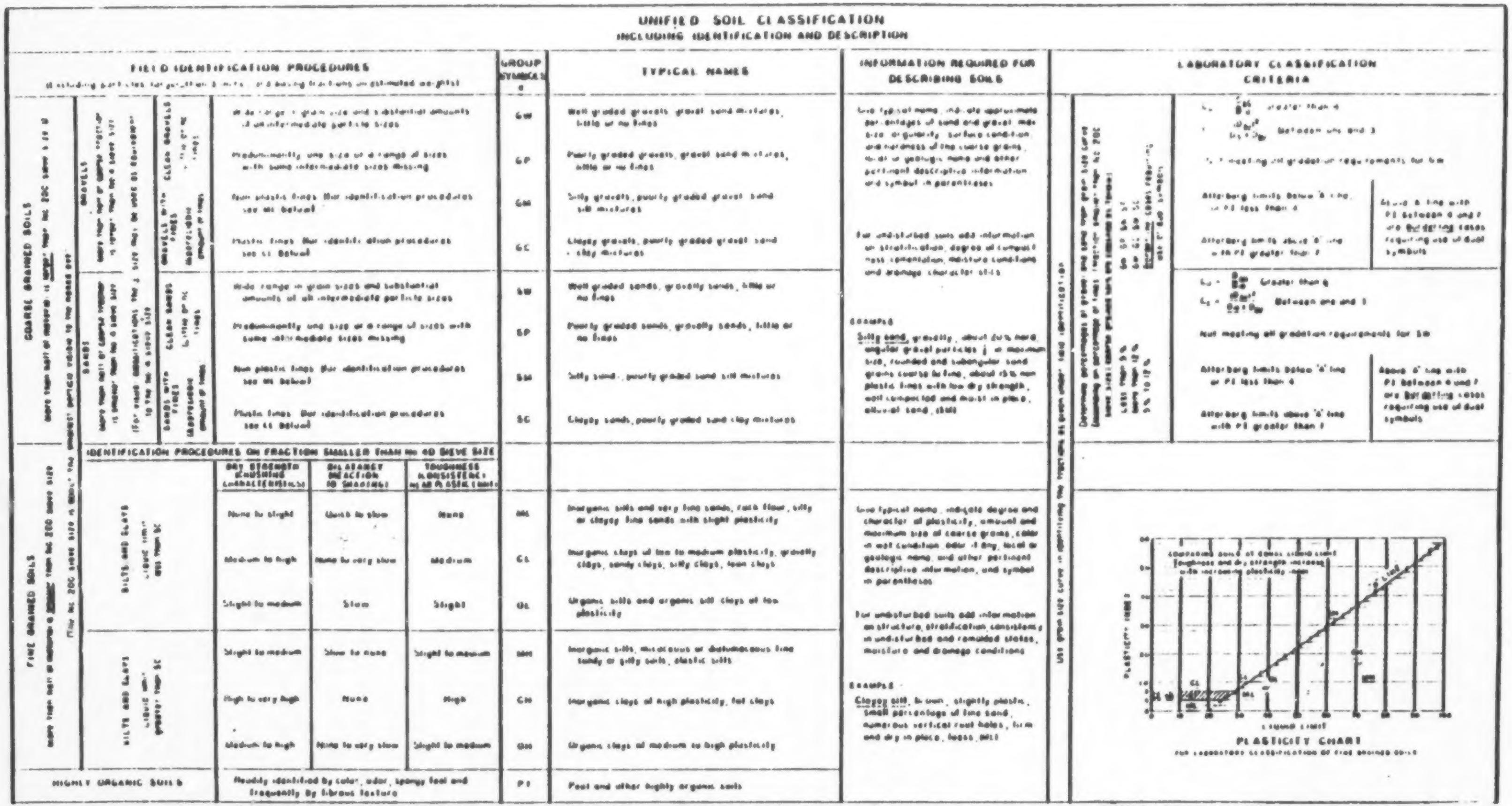
Except for small editorial changes, the following sections (Sections 3.4.2 through 3.4.2.3.3) are taken directly from the Earth Manual (U.S. Department of Interior, 1974).

Soils in nature seldom exist separately as gravel, sand, silt, clay, or organic matter, but are usually found as mixtures with varying proportions of these components. The Unified Soil Classification System (USCS) is based on recognition of the type and predominance of the constituents, considering grain size, gradation, plasticity, and compressibility. It divides soils into three major divisions: coarse-grained soils, fine-grained soils, and highly organic (peaty) soils. In the field, identification is accomplished by visual examination for the coarse grains and by a few simple hand tests for the fine-grained soils or fraction, as described in the classification chart (Figure 3-9 and fully described in Section 3.4.2.3). In the laboratory, the grain-size curve and the Atterberg limits can be used in addition to visual examination. The peaty soils (Pt) are readily identified by color, odor, spongy feel, and fibrous texture and are not further subdivided in the classification system.

3.4.2.1 Field Classification--

A representative sample of soil (excluding particles larger than 3 inches) is first classified as coarse grained or fine grained by estimating whether 50 percent, by weight, of the particles can be seen individually by the naked eye. Soils containing more than 50 percent of individual particles that can be seen are coarse-grained soils; soils containing more than 50 percent of particles smaller than the eye can individually distinguish are fine-grained soils. (The No. 200 sieve size [74 μm] particles are about the smallest that can be seen individually by the unaided eye.) If the soil is predominantly coarse grained, it is then identified as being a gravel or a sand by estimating whether 50 percent or more, by weight, of the coarse grains are larger or smaller than the No. 4 sieve size (about 1/4 inch).

If the soil is a gravel, it is next identified as being "clean" (containing few or no fines) or "dirty" (containing an appreciable amount of fines). For clean gravels, final classification is made by estimating the gradation: the well-graded gravels belong to the GW group, and uniform and



UNIFIED SOIL CLASSIFICATION
INCLUDING IDENTIFICATION AND DESCRIPTION

GROUP SYMBOLS	TYPICAL NAMES
GW	Well graded gravels, gravel sand mixtures, little or no fines
GP	Poorly graded gravels, gravel sand mixtures, little or no fines
GM	Silty gravels, poorly graded gravel sand silt mixtures
GC	Clayey gravels, poorly graded gravel sand clay mixtures
SW	Well graded sands, gravelly sands, little or no fines
SP	Poorly graded sands, gravelly sands, little or no fines
SM	Silty sands, poorly graded sand silt mixtures
SC	Clayey sands, poorly graded sand clay mixtures
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
OL	Organic silts and organic silt clays of low plasticity
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	Inorganic clays of high plasticity, fat clays
OH	Organic clays of medium to high plasticity
Pt	Peat and other highly organic soils

IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE

DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	GROUP SYMBOLS
None to slight	Quick to slow	None	ML
Medium to high	None to very slow	Medium	CL
Slight to medium	Slow	Slight	OL
Slight to medium	Slow to none	Slight to medium	MH
High to very high	None	High	CH
Medium to high	None to very slow	Slight to medium	OH

HIGHLY ORGANIC SOILS: Readily identified by color, odor, spongy feel and frequently by fibrous texture

INFORMATION REQUIRED FOR DESCRIBING SOILS

Give typical name; indicate approximate percentages of sand and gravel, max. size, angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.

For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.

EXAMPLE:
Silty sand, gravelly; about 20% hard, angular gravel particles ½ in. maximum size; rounded and subangular sand grains coarse to fine; about 15% non plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).

Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet condition; odor, if any; local or geologic name, and other pertinent descriptive information; and symbol in parentheses.

For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.

EXAMPLE:
Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML).

PLASTICITY CHART

a Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel sand mixture with clay binder.
b All sieve sizes on this chart are U.S. standard.

ADOPTED BY CORPS OF ENGINEERS AND BUREAU OF RECLAMATION, JANUARY 1952

Source: U.S. Department of Interior, 1974

Figure 3-9. Unified soil classification chart.

skip-graded gravels, the poorly graded gravels, belong to the GP group. Dirty gravels are of two types: those with nonplastic (silty) fines (GM) and those with plastic (clayey) fine (GC). The determination of whether the fines are silty or clayey is made by three manual tests for fine-grained soils (described in Section 3.5.2.3).

If a soil is a sand, the same steps and criteria are used as for the gravels to determine whether the soil is a well-graded clean sand (SW), poorly graded clean sand (SP), sand with silty fines (SM), or sand with clayey fines (SC).

If a material is predominantly (more than 50 percent by weight) fine-grained, it is classified into one of six groups (ML, CL, OL, MH, CH, OH) by estimating its dilatancy (reaction to shaking), dry strength (crushing characteristics), and toughness (consistency near the plastic limit) and by identifying it as being organic or inorganic. (The test procedures and behavior of the various groups of fine-grained soils for each of the hand tests are shown on the classification chart (Figure 3-8) and are described in Section 3.4.2.3).

Soils typical of the various groups are readily classified by the foregoing procedures. Many natural soils, however, will have property characteristics of two groups because they are close to the borderline between the groups either in percentages of the various sizes or in plasticity characteristics. For this substantial number of soils, borderline classifications are used; i.e., the two group symbols most nearly describing the soil are connected by a hyphen, such as GW-GC.

If the percentages of gravel and sand sizes in a coarse-grained soil are nearly equal, the classification procedure is to assume the soil is a gravel and then continue the classification procedure using the chart until the final soil group, say GC, is reached. Since it could have been assumed that the soil is a sand, the correct field classification is GC-SC because the criteria for the gravel and sand subgroups are identical. Similarly, within the gravel or sand groupings, borderline classifications such as GW-GP, GM-GC, GW-GM, SW-SP, SM-SC, and SW-SM can occur.

Proper classification of a soil near the borderline between coarse-grained and fine-grained soils is accomplished by classifying it first as a coarse-grained soil and then as a fine-grained soil. Such classifications as SM-ML and SC-CL are common.

Within the fine-grained division, borderline classifications can occur between low-liquid-limit soils and high-liquid-limit soils as well as between silty and clayey materials in the same range of liquid limits. For example, one may find ML-MH, CL-CH, and OL-OH soils; ML-CL, ML-OL, and CL-OL soils; and MH-CH, MH-OH, and CH-OH soils.

3.4.2.2 Laboratory Classification--

Although most classifications of soil will be done visually and by simple hand tests, the USCS has provided for precise delineation of the soil groups by gradation analyses and Atterberg limits tests in the laboratory. Laboratory classifications are often performed on representative samples of soils which are to be subjected to extensive testing for shear strength, compressibility, and permeability. They can also be used to advantage in training the field classifier of soils to improve his ability to estimate percentages and degrees of plasticity.

The grain-size curve is used to classify the soil as being coarse-grained or fine-grained and, if coarse-grained, into gravel or sand using the 50-percent criterion. Within the gravel or sand groupings, soils containing less than 5 percent finer than the No. 200 sieve size are considered "clean" and are then classified as well-graded or poorly graded by their coefficients of uniformity and of curvature. In order for a clean gravel to be well-graded (GW), it must have both a coefficient of uniformity, C_μ, greater than 4 and a coefficient of curvature, C_c, between 1 and 3; otherwise, it is classified as a poorly graded gravel (GP). A clean sand having both C_μ greater than 6 and C_c between 1 and 3 is in the SW group; otherwise, it is a poorly graded sand (SP).

"Dirty" gravels or sands are those containing more than 12 percent fines, and they are classified as either silty or clayey by results of their Atterberg limits tests as plotted on the plasticity chart shown in Figure 3-8. Silty fines are those that have a PI less than 4 or that plot below the "A" line. Clayey fines are those that have a PI greater than 7 and that plot above the "A" line.

Coarse-grained soils containing between 5 and 12 percent fines are borderline cases between the clean and dirty gravels or sands (i.e., GS, GP, SW, SP, GM, GC, SM, and SC). Similarly, borderline cases may occur in dirty gravels and dirty sands where the PI is between 4 and 7 (GM-GC, SM-SC). It is theoretically possible, therefore, to have a borderline case of a borderline case; but this refinement is not used, and the rule for correct classification is to favor the nonplastic one. For example, a gravel with 10 percent fines, a C_μ of 20, a C_c of 2.0, and a plasticity index of 6, would be classified GW-GM rather than GW-GC.

If a soil is determined to be fine-grained by using the grain-size curve, it is further classified into one of the six groups by plotting the results of Atterberg limits tests on the plasticity chart, with attention being given to the organic content. Inorganic, fine-grained soils with a plasticity index greater than 7 and above the "A" line are CL or CH, depending on whether their liquid limits are less than 50 percent or more than 50 percent, respectively. Similarly, inorganic, fine-grained soils with a plasticity index less than 4 or below the "A" line are ML or MH, depending on whether their liquid limits are less than or more than 50 percent, respectively. Fine-grained soils that fall above the "A" line but that have a plasticity index between 4 and 7 are classified as ML-CL.

Soils below the "A" line that are definitely organic are classified as OL if they have liquid limits less than 50 percent and as OH if the liquid limits are above 50 percent. Organic silts and clays are usually distinguished from inorganic silts, which have the same position on the plasticity chart, by odor and by color. However, when the organic content is doubtful, the material can be oven dried, remixed with water, and retested for liquid limits. The plasticity of fine-grained organic soils is greatly reduced on oven drying due to irreversible changes in the organic colloids. Oven drying also affects the liquid limit of inorganic soils but to a much smaller degree. A reduction in liquid limit after oven drying to a value less than three-fourths of the liquid limit before oven drying is considered positive identification of organic soils.

3.4.2.3 Field Identification Procedures for Fine-Grained Soils or Fractions--These procedures are to be performed on the minus No. 40 sieve size particles (approximately 1/64 inch). For field classification purposes, screening is not intended; simply remove by hand the coarse particles that interfere with the tests.

3.4.2.3.1 Dilatancy (Reaction to Shaking)--After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat, which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction, whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

3.4.2.3.2 Dry Strength (Crushing Characteristics)--After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty, whereas a typical silt has the smooth feel of flour.

3.4.2.3.3 Toughness (Consistency Near Plastic Limit)--After removal of particles larger than the No. 40 sieve size, a specimen of soil about one-half cubic inch in size is molded to the consistency of putty. If too dry, water must be added and, if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the speciment stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity or materials such as kaolin-type clays and organic clays that occur below the A-line.

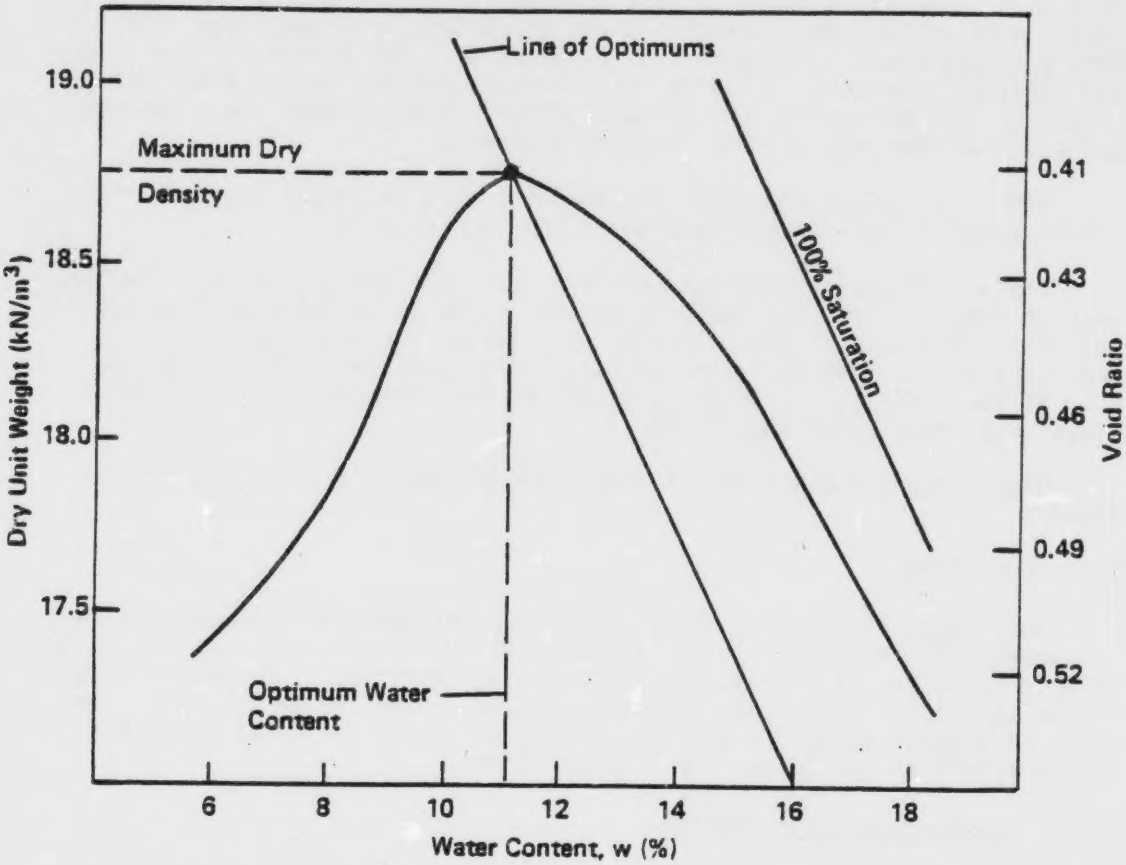
Highly organic clays have a very weak and spongy feel at the plastic limit.

3.5 COMPACTION

Soil compaction is the rapid application of mechanical force to a soil to increase its density. In the construction of clay liners, compaction is done to decrease the permeability of the liner material. Compactive effort is the term used to describe the amount of force applied to the soil. The units of compactive effort are foot pounds of energy per cubic foot of soil (ft-lb f/ft^3) or joules per cubic meter (J/m^3).

3.5.1 Fundamentals of Compaction

In a 1933 series of publications on soil compaction, R. R. Proctor described a laboratory procedure that, in its modern form, has become the standard method for determining the moisture, dry density, and compactive effort relationship of compacted soils (Proctor, 1933). In the Proctor compaction test, soil densification is achieved through the application of a standard dynamic impact. A standard method has been adopted and described in ASTM standard test method D698-78 (ASTM, 1983). The test is performed by placing 1 inch of test soil in a cylindrical compaction mold and dropping a 5.5-pound weight onto its surface 25 times from a height of 1 foot. This procedure is repeated for two subsequent 1-inch lifts (soil layers), resulting in a compacted sample with three lifts. The dry density of the compacted sample is then determined. This procedure is repeated several times on samples of the same soil at different moisture contents. When the testing is completed, the water contents of the samples are plotted against the dry-densities achieved after compaction. This exercise produces a compaction curve (also called a moisture density curve) as illustrated in Figure 3-10.



Source: Lambe, 1955

Figure 3-10. Typical soil compaction curve illustrating maximum dry density and optimum water content.

The illustration presents only one curve achieved with one compactive effort. Changing the compactive effort will produce similar curves with different optimums, with increasing compactive effort increasing the maximum dry density and decreasing the optimum moisture content. A typical series of compaction curves for a single soil at different compactive efforts is presented in Figure 3-11. Note that changing the compactive effort also can change the shape of the compaction curve.

The most important feature on the compaction curve is its peak, which represents the maximum dry density that can be achieved with a given compactive effort. The moisture content corresponding to this peak is called the optimum water content. The line of optimums connects the maximum densities and optimum water contents of compaction curves produced by different compactive efforts applied to samples of the same soil. The line of optimums runs nearly parallel to the zero air voids line (also called the 100-percent saturation line), which represents the saturated water contents for different dry densities of a given soil.

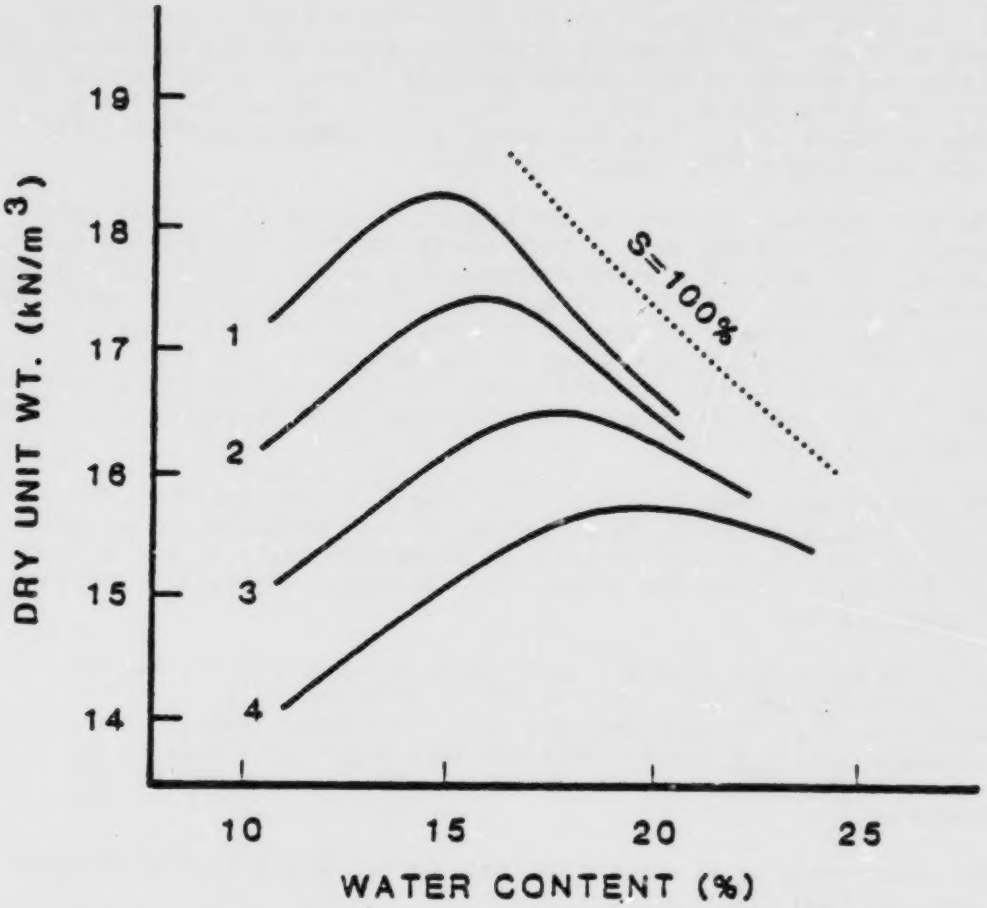
The compaction curves in Figure 3-11 clearly illustrate that, for a given soil, compacted density is a function of soil moisture content and compactive effort. Care must be taken in compaction tests to ensure that moisture is evenly distributed through the soil mass. Winterkorn and Fang (1975) point out that:

> . . . to avoid irregular and meaningless moisture density curves in laboratory and field tests on highly cohesive soils, it is imperative that the moisture be evenly distributed throughout the secondary soil aggregates. This may take from 1 to 7 days in the case of highly cohesive soils to which water is added in the dry condition.

The interpretation of compaction tests is not always straightforward. Winterkorn and Fang (1975) point out:

> . . . even properly performed tests may not yield the generally expected paraboloid curves. Andrews et al. (1967) found two optimum moisture contents for the compaction of Na-bentonite, one at 100 percent H_2O yielding 73 lb/ft^3 and one at 50 percent H_2O yielding 65 lb/ft^3. Lee and Suedkamp (1972) reported the existence of four types of moisture-density curves [as shown in Figure 3-12]. They are (A) single-peak, (B) 1½ peak, (C) double peak, and (D) oddly shaped with no distinct optimum moisture content. They related the different types to the liquid limit ranges as follows: LL<30, double and 1½ peaks; LL 30 to 70, typical single peak; LL>70, peaked and oddly shaped curves.

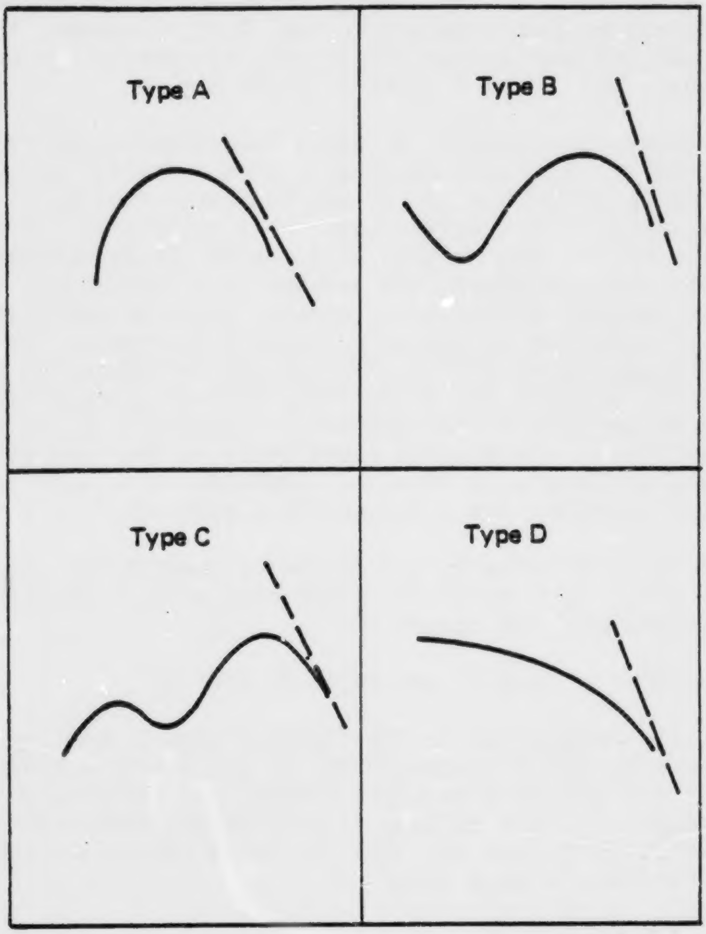
These unusual curves are not likely to be encountered with clays suitable for liners (Mitchell, J. K., 1985, Dept. of Civil Engineering, University of California, Berkeley, personal communication).



No.	Layers	Blows per Layers	Hammer Mass	Hammer Drop	
1	5	55	4.54 kg	457 mm	(mod. AASHO)
2	5	26	5.54	457	
3	5	12	4.54	457	(std. AASHO)
4	3	25	2.50	305	

Source: Lambe and Whitman, 1979

Figure 3-11. Compaction curves for different compactive efforts applied to a silty clay.



After Winterkorn and Fang, 1975

Figure 3-12. Four types of compaction curves found from laboratory investigation.

3.5.2 Compaction and Permeability

During installation of a clay liner, compaction is controlled by measuring density and moisture content in each lift. However, these measurements by themselves are meaningless unless they ultimately can be related back to the permeability. (See Figures 3-13 and 3-14.)

Mitchell, Hooper, and Campanella (1965) investigated the relationship between these variables and found that, at a given density, permeability was very sensitive to variations in the molding water (moisture) content. They performed a series of compaction tests on a silty clay, samples of which were kneaded to constant density at different water contents. When permeability tests were performed, the permeability increased slightly between 12 and 18 percent molding water content but then decreased by around 3 orders of magnitude as the water content increased from 18 to 19.5 percent. The decreased permeability corresponded to compaction performed at water contents on the wet side of optimum; minimum permeability occurred on the "wet-of-optimum" side of the compaction curve and not at the maximum density. Mitchell et al. (1965) also found that, in the laboratory, lower permeabilities are achieved with kneading compaction than with static compaction. These phenomena are discussed in Chapter 2 of this report.

This discussion gives rise to the following conclusion: to achieve a specified permeability, soil moisture, compactive effort, and dry density must be carefully measured and controlled.

3.6 FIELD MEASUREMENT OF DENSITY AND MOISTURE CONTENT

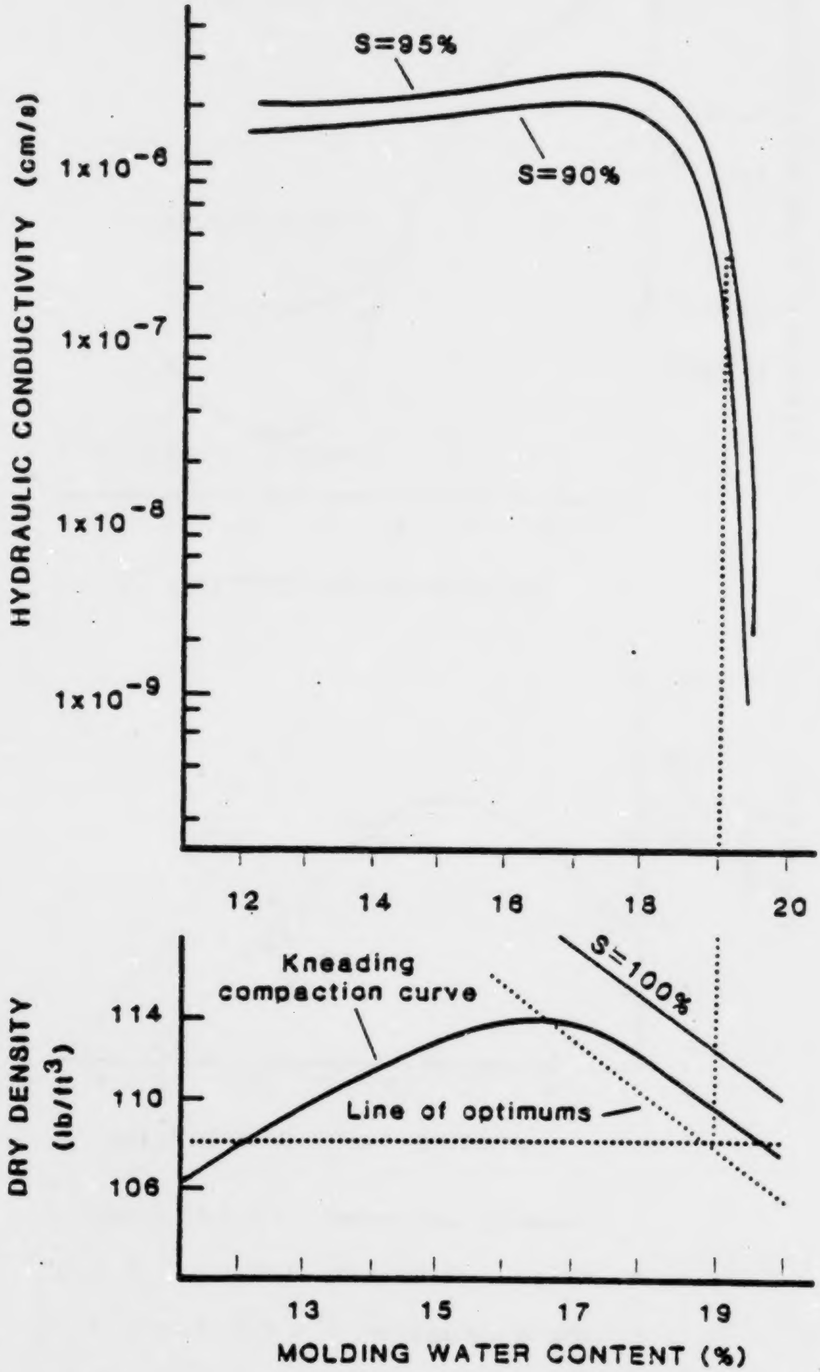
Controlling clay compaction in the field so that a specified permeability is achieved is done through control of the moisture content and density. Methods for measuring moisture content are summarized in Appendix A, Methods 1 through 7. Field density or unit weight measurements are summarized in Methods 8 through 13. The following text discusses the more commonly used techniques in more detail.

3.6.1 Traditional Methods

3.6.1.1 Density--

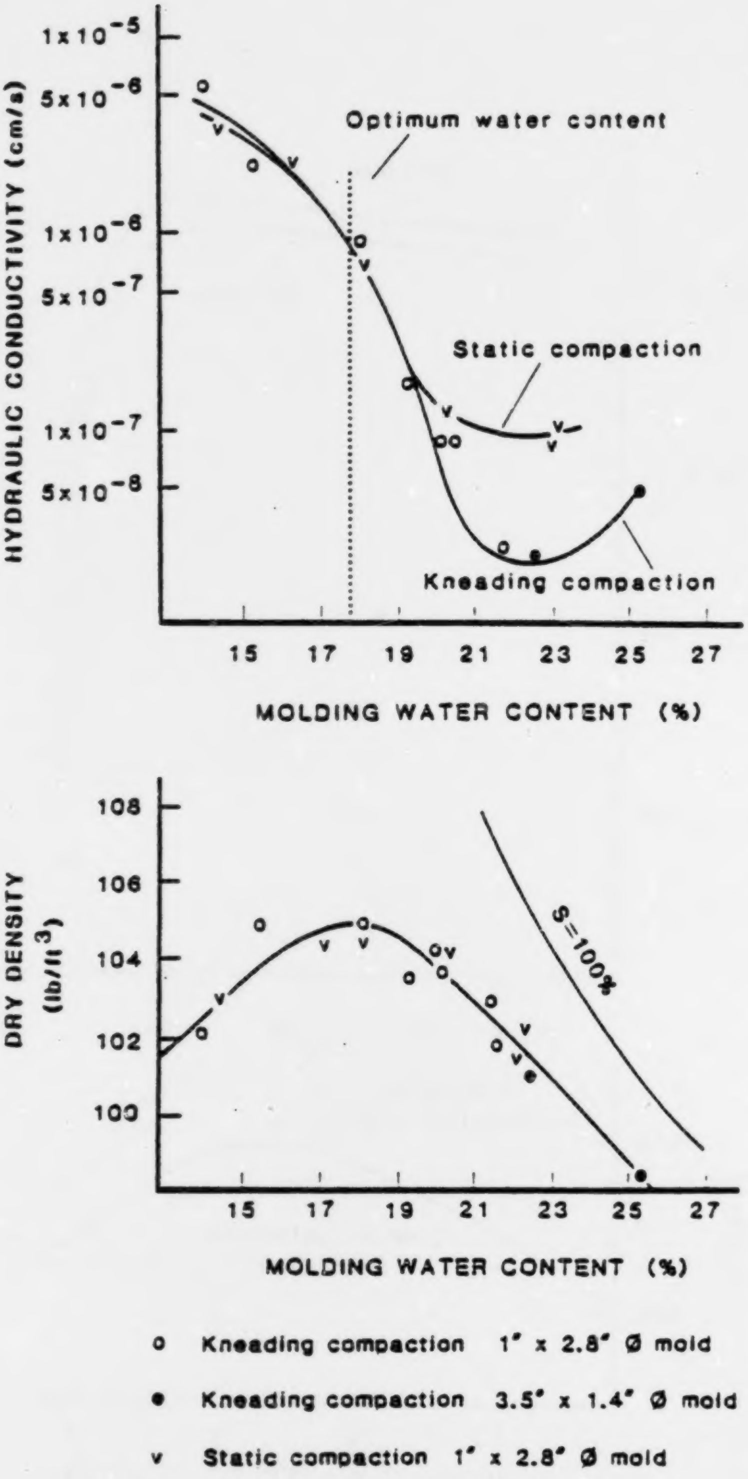
Two traditional methods are used for measuring density in the field. In one type of test, a small hole is dug in the compacted fill and the excavated material is saved and weighed. The volume of the hole is measured by filling it with sand or liquid with a device that measures the amount of material required to fill the hole. The sand cone and rubber balloon methods are examples of this category of test (Appendix A, Methods 10 and 11).

Another technique is to drive a hollow cylinder into the fill, remove a core, trim it to a known volume, and then determine its weight. This drive-cylinder method (Appendix A, Method 12) and the sand cone and rubber balloon methods take time because the sample must be oven dried before the dry density can be determined.



Source: Mitchell, 1976

Figure 3-13. Permeability as a function of molding water content for samples of silty clay prepared to constant density by kneading compaction.



Source: Mitchell, 1976

Figure 3-14. Influence of the method of compaction on the permeability of silty clay.

3.6.2 Nuclear Methods

Nuclear gauges offer a faster and more convenient method for measuring field density and moisture content than those methods described in Section 3.6.1 and are presently widely used for earthwork compaction quality control. Nuclear gauges are designed to give very rapid measurements of density and moisture content.

3.6.2.1 Nuclear Density Gauge--

Nuclear density gauges consist of a source of gamma rays, typically Cs-137 (0.662 Mev), and a detector (or nest of detectors) separated horizontally from the source. The gauges are used in two modes: a "back-scattering" mode in which the gamma rays are directed, by collimation, downward into the soil where they are scattered with some fraction reaching the detector and a transmission mode in which the source is lowered into a hole in the soil (2 to 12 inches deep) and the number of gamma rays penetrating the soil to reach the detector on the surface is recorded. In the range of densities of interest (70 to 160 lb/ft^3), the gamma rays that reach the detector decrease with increasing soil density.

Gamma rays interact with the electrons that are a part of the atoms making up the soil. Two types of interactions are exploited for measuring density. The primary interaction is Comptom scattering wherein the gamma ray is deflected (scattered) by an electron and continues in a different direction at a reduced energy. For the elements found in soil, Compton scattering is the predominant reaction in the energy range above 100 keV. The second interaction is photoelectric absorption wherein the gamma ray gives up all its energy to an electron and is removed from the beam. The frequency of this interaction becomes greater as the gamma ray energy decreases from 100 keV. The probability of this reaction also increases with the atomic number of the element.

The relative frequency of these two interactions is a function of the soil's chemical composition. The effects can best be illustrated by first considering the predominant reaction, Compton scattering, and then the modification introduced by photoelectric absorption.

The Compton scattering cross section, i.e., the probability of a scattering event, is proportional to the electron density of the medium. This is given by:

$$D_e = D_m N \sum_i W_i Z_i/A_i \quad , \qquad (3.11)$$

where

D_m = mass density of the medium

N = Avogadro's number

W_i = weight fraction of element

Z_i = atomic number of element

A_i = atomic weight of element.

Thus, the electron density is related to the mass density by the factor Z/A of the medium. The Z/A of the various soil components is given in Table 3-3. Except for Fe_2O_3 (Z/A = 0:476), the major soil components have Z/A values that fall within the range of 0.489 to 0.500. Thus, if Compton scattering alone were responsible for the attenuation of the gamma rays, the mass densities would be within a few tenths of a percent of the instrument gauge indications except for soils containing appreciable amounts of Fe_2O_3. Water also has a measurable effect on nuclear density measurements. Because of its high Z/A, its electron density (per gram) is high. As a result, a dry soil of a given density will produce a slightly lower nuclear gauge density reading than a wet soil of the same mass density. The different readings are a consequence of the difference in the Z/A of the two soils (Table 3-3; clay and clay + 10 percent H_2O).

Thus, with respect to the Compton scattering component, the gauge will read iron-rich soils lower than their actual density and damp soils somewhat higher than their actual density.

The cross section for the photoelectric effect interaction of gamma rays with electrons is experimentally determined to be about proportional to Z^5/A. Thus, this effect becomes increasingly important for higher Z elements such as calcium, which has a Z of 20. The other major earth elements--oxygen, silicon, and aluminum--have relatively low Z values (8, 14, and 13, respectively), so the photoelectric effect for these is less than for calcium. The consequence is that more gamma rays are absorbed per unit of electron density in calcareous soils than in noncalcareous soils and the nuclear gauge density reading is therefore higher than the actual density.

To minimize this effect, ASTM Standard 2922 (ASTM 1983) requires that nuclear density gauge calibration blocks include two that bracket the attenuation coefficient of soils and suggests limestone and granite. The calibration curve is adjusted to fall halfway between the limestone and granite data. As a result of this compromise, highly calcareous soils will read 2 to 3 lb/ft^3 high and highly siliceous soils will read 2 to 3 lb/ft^3 low when measurements are made in the transmission mode. This effect is exaggerated further in the backscattering mode, where the photoelectric effect is greater. In the backscattering mode, highly calcareous soils will read 3 to 4 lb/ft^3 high and highly siliceous soils will read 3 to 4 lb/ft^3 low.

Although iron is not a major component of the earth's crust, some soils contain sufficient iron to affect the reading of a nuclear density gauge. Biotite mica, other iron silicates, and iron oxides (e.g., goethite and hematite) cure some of the more common iron-containing minerals in soils; nuclear density measurements in soils containing iron minerals should be made with care. Because the Z of iron is 26, the photoelectric effect becomes quite important and the gauge will indicate a density higher than the actual value when calibrated according to ASTM Standard 2922.

TABLE 3-3. Z/A OF VARIOUS SOIL COMPONENTS

Components	Z_i/A_i
Silica (SiO_2)	0.500
Feldspar ($KAlSi_3O_8$)	0.495
Lime ($CaCO_3$)	0.499
Alumina (Al_2O_3)	0.490
Soda (Na_2O_3)	0.489
Magnesia (MgO)	0.495
Clay ($Al_6Si_2O_{13}$)	0.496
Clay + 10% H_2O	0.502
Fe_2O_3	0.476
H_2O	0.555

A method for overcoming the chemical composition error is to calibrate the gauge against a series of sand cone tests performed on the soil of interest. The precision of a series of sand cone tests is poorer than the precision of nuclear gauge tests, but the accuracy is generally better if the sand cone tests are done carefully. The ratio of the sand cone density to the nuclear density may then be applied to the nuclear gauge reading to achieve the corrected density.

When used in the backscattering mode, "surface roughness" may cause inaccurate density measurements. If the soil surface is not a perfect plane, air voids are formed between the gauge and the soil, reducing the density of the volume beneath the gauge. A 50-mil air gap will result in a density reading that is about 4 lb/ft^3 low.

The uniformity of the soil density is also important in the backscattering mode. About 60 percent of the gauge response is from the top inch of soil, 25 percent from the second inch, and 10 percent from the third inch. Thus, while a gauge may be quoted as having 95 percent of its response from a 3-inch depth, the response versus depth in this region is in no way uniform. In summary:

- Nuclear density gauges are factory calibrated to give compromised measurements in two soil extremes. In highly calcareous dry soils, the gauge will read high by 2 to 3 lb/ft^3 at 120 lb/ft^3 in the transmission mode and in highly siliceous soils low by 2 to 3 lb/ft^3. In the backscattering mode, the gauges are 3 to 4 lb/ft^3 high and low, respectively, at 120 lb/ft^3. The gauge would be expected to read properly in clays with up to 10 to 12 percent water content if no high Z elements such as calcium and iron are present in appreciable quantities. Nuclear gauges will read iron-rich soils lower than their actual density and damp soils somewhat higher than their actual density.

- In the backscattering mode, the nuclear gauge is sensitive to any density variations in the upper 3 inches of soil and to the flatness of the soil surface upon which the gauge rests.

- It is customary to calibrate the nuclear gauge against a series of sand cone tests to overcome chemical composition problems in the field. A gauge reading (preferably in the transmission mode) is taken, and then a sand cone test is made on the same volume measured by the gauge. The average of the ratio of the sand cone test to the gauge reading for a series of tests then becomes the correction factor to be applied to the gauge reading. This correction factor will apply as long as the chemical composition of the soil does not change appreciably.

3.6.2.2 Nuclear Moisture Gauge--

Nuclear moisture gauges consist of a source of fast neutrons (usually an americium-beryllium source) and a slow neutron detector mounted as close to one another as possible. The principle of operation is that of neutron moderation, i.e., the slowing down of neutrons caused by elastic scattering from the nuclei in the scattering medium. Hydrogen is an excellent moderator because its nuclear mass is the same as the mass of a neutron and each interaction with hydrogen results in a major energy loss to the neutron. On the average, about 19 collisions with hydrogen nuclei will thermalize the neutron, i.e., reduce its energy to that of the surrounding nuclei. The other elements in the earth's crust are not nearly as effective--oxygen requires about 158 collisions on the average and silicon 273. The readings obtained with a nuclear moisture gauge are essentially independent of the density of the medium because, except for hydrogen, the medium is nearly transparent to the neutrons.

By using a slow neutron detector, i.e., one responsive only to thermal neutrons, one can measure the number of neutrons themalized, which is a function of the number of hydrogen atoms in the medium. In inorganic soils, water provides the bulk of the hydrogen and thus the water content can be measured. The greater the water content, the greater the number of neutrons thermalized.

Because of the system's geometry, the gauge is most sensitive to the hydrogen content in the immediate vicinity of the detector. At low moisture contents, the neutron moves further from the source-detector position before it has undergone enough collisions to thermalize it and is far enough from the detector that the probability of returning to the detector is low. On the other hand, a high moisture content will permit thermalization much closer to the detector. This leads to the conclusion that the active volume of measurement is a function of the moisture content.

Experimentally determined depths of measurement for a typical neutron gauge give the following expression for the depth where 95 percent of the detected interactions will occur:

$$D \text{ (inches)} = 11 - 0.17 \text{ MC} \quad , \qquad (3.12)$$

where

MC = moisture content in pounds of water per cubic foot of soil.

When Equation (3.12) is applied to a hypothetical case, if a soil sample has a density of 120 lb/ft^3 and moisture content of 12 percent, it contains 14.4 pounds of water per cubic foot and the sensitive depth is 8.5 inches. Note, however, that if the moisture is nonuniform, the gauge response will be biased toward the moisture content near the detector.

Another reaction can modify the gauge reading when certain soil elements are present. Aside from the moderation interactions, certain elements have a high probability of absorbing a neutron. The only ones normally found in soil are iron and chlorine, the latter where saline soils exist. Thus, coastal soils may contain sufficient chlorine to be significant and iron oxide or silicate contents of 35 to 40 percent will cause errors. In summary:

- Neutron gauges are useful for measuring the moisture content of inorganic soils of uniform water content. Although the sensitive depth of measurement may normally be 6 to 8 inches, the response is biased toward the moisture content near the detector.

- If hydrogen-containing materials (including clay minerals) other than water are present, the gauge will respond to this hydrogen. Coastal soils may have sufficient chlorine content to interfere, and high iron oxide or silicate content will influence the moisture measurement.

- In order to compensate for the soil compositions that may affect the neutron response, it is customary to calibrate the gauge against tests on the oven-dry soil. This is generally done with the soil samples extracted for the density calibration.

3.6.2.3 Modern Nuclear Density and Moisture Gauges--

Current nuclear gauges contain both density and moisture-measuring components. A key pad, display unit, and microprocessor enable various correction factors to be entered into the gauge for particular soils. Operating manuals provide complete instructions for the use of the gauge under various conditions. In addition, gauge manufacturers provide training seminars on the proper use of their gauges, including safety considerations.

3.7 TESTING FOR SHEAR STRENGTH

The shear strength of a soil must be known before an earthen structure can be designed and built with assurance that the slopes will not fail (see Chapter 5).

A soil mass may be considered to be a compressible skeleton of solid particles. In saturated soils the void spaces are completely filled with water; in partially saturated soils the void spaces are filled with both water and air. Shear stresses are carried only by the skeleton of solid particles, whereas the normal stress on any plane is carried by both the solid particles and the pore water.

The development of shear in a saturated cohesive (clay) soil is complex because an applied load is initially supported by the stress in the pore water (neutral stress) and is not immediately transmitted to the soil structure. Because of the very low permeability of clays, neutral stresses are transmitted to the soil structure very slowly, sometimes requiring months or years before the soil structure feels the full stress increase. Another complicating factor is the attractive and repulsive interaction between the clay particles (Sowers and Sowers, 1971).

The triaxial compression test is commonly used to measure the shear strength of a soil under controlled drainage conditions. A triaxial compression chamber consists primarily of a head plate and a base plate separated by a transparent plastic cylinder (Figure 3-15). In the basic triaxial test, a cylindrical specimen of soil, encased in a rubber membrane, is placed in the chamber. Connections at the ends of the soil specimen permit controlled drainage of pore water. The chamber is filled with water and pressurized, thereby establishing a confining pressure on the specimen. The specimen is then loaded axially to the point of failure. The axially applied stress is called the "deviator stress." In general, a minimum of three specimens, each under a different confining pressure, are tested to establish the relation between shear strength and normal stress (U.S. Department of the Army, 1970).

Three types of triaxial compression tests have been developed to measure the shear strength of clays. The tests differ in the drainage conditions established for the sample and shear stress application rate. These tests are:

- Consolidated-Drained (CD) Test or Slow (S) Test--The desired confining stress is applied to the specimen and drainage is allowed until 100-percent consolidation is achieved. The rate of applied shear stress is slow enough that essentially no change in the initial pore pressure occurs.

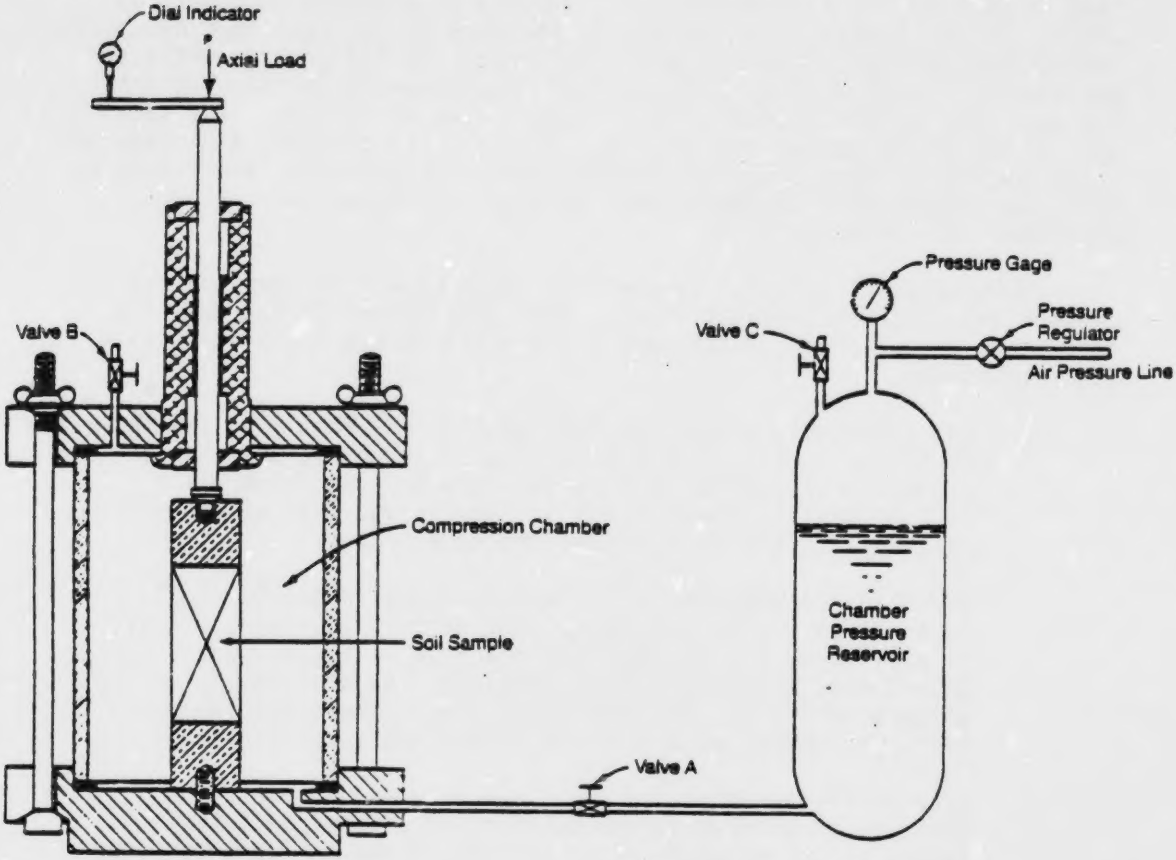
- Consolidated-Undrained (CU) Test or Consolidated Quick (R) Test--As in the CD test above, the derived confining stress is applied to the specimen and drainage is allowed until 100-percent consolidation is achieved. No drainage is allowed during the application of shear stress and the specimen remains at a constant volume and water content.

- Unconsolidated-Undrained (UU) Test or Quick (Q) Test--No drainage is allowed during application of the confining stress or during application of shear stress. The unconfined compression test is a special case of the UU test with the confining stress equal to zero. The principal stress difference at failure is called the unconfined compressive strength.

Detailed discussion of the protocols for shear strength testing and interpretation of the test results are beyond the scope of this document. The reader is referred to the U.S. Department of Interior (1974) and U.S. Department of the Army (1970) for test protocols. Explanations of the theory and interpretation of the test can be found in geotechnical engineering texts such as Lambe and Whitman (1979) and Holtz and Kovacs (1981).

3.8 HYDRAULIC CONDUCTIVITY TESTING

Laboratory and field tests to quantify the hydraulic conductivity (permeability) of low-porosity soils to water have been used for years. Historically, information on permeability has been required to evaluate seepage characteristics of soils in a variety of applications ranging from earthen dams to septic tank drainage fields. These same test procedures



Source: U.S. Department of the Army, 1970

Figure 3-15. Schematic diagram of triaxial compression apparatus for Q test.

have been adapted, with some modifications, to evaluate the performance of compacted clay liners.

Hydraulic conductivity testing of clay liner material is used for facility design, for construction quality control (CQC), and for clay/ chemical compatibility evaluation. CQC tests are conducted to ensure that the hydraulic conductivity of an installed clay liner meets the performance specification. A compatibility test is conducted to determine if a particular soil's permeability is altered by a liquid waste with which it might come in contact in a specific application. Chemical compatibility is discussed, at length, in Chapter 4.

Theoretical knowledge has not yet reached the stage of providing reliable estimates of permeability. Thus, field and laboratory measurements are needed. Standard procedures, however, have not been adopted for permeability testing related to hazardous waste containment.

Effective porosity, pore-size distribution, fluid viscosity, and fluid density determine the permeability of clay soils to fluids. Any action that affects the soil's fabric may affect the effective porosity and, in turn, the permeability. Such modification of the soil fabric might be physical, as with soil compaction, or chemical, through the various processes that can occur when a chemical waste interacts with the clay (see Chapter 4).

In general, the procedure for measuring the permeability of a compacted soil is to enclose the sample tightly in a cylinder (permeameter) and then to pass the liquid (permeant), usually under pressure, through the sample. The pressure differential across the sample is expressed in terms of hydraulic gradient (a dimensionless quantity) that is the pressure drop across the sample divided by the sample's height. The gradient can be controlled by superimposing air pressure above the permeant supplied to the sample and by regulating the pressure applied at the effluent end of the column. High hydraulic gradients reduce testing time by forcing permeant through the sample at a greater rate than could be achieved with low gradients. However, the amount of gradient may influence the test results.

3.8.1 Darcy's Law

Hydraulic conductivity (K) or permeability is a measure of how rapidly a permeant fluid can move through porous soils under a hydraulic gradient. Quantitatively, K is defined as the constant of proportionality in Darcy's law (Equation 3.14). To calculate K for a given soil sample, one must be able to measure the volumetric flow rate through the sample, the cross-sectional area of the sample perpendicular to the direction of fluid flow, and the hydraulic gradient across the sample.

The flow of liquids through clay has been found to obey Darcy's Law, which states that the volumetric flow rate of water through a porous medium is proportional to the cross-sectional area of the medium and to the hydraulic gradient impressed across it. The constant of proportionality (K) is defined as the soil's hydraulic conductivity. This is expressed in Equation (3-13):

$$Q = KAh/L \quad , \qquad (3.13)$$

where

Q = volumetric flow rate, cm^3/s

K = hydraulic conductivity (permeability), cm/s

A = cross-sectional area of specimen, cm^2

h = change in hydraulic head across the specimen, cm

L = length of sample, cm.

Note that h/L is the hydraulic gradient. Also note that although K appears to have the dimensions of a velocity, this is an artifact due to the cancellation of units. The true dimensions are cm^3/cm^2 s (i.e., volume per unit area per unit time). Darcy's Law assumes a direct proportionality between the hydraulic gradient and the flow rate.

When fluids other than water are used, the permeability as defined above will be different if the viscosity and/or density of the fluid differs from that of water. It is convenient to define an intrinsic permeability coefficient that considers these two parameters. The equation for intrinsic permeability is:

$$k = K\mu/\rho g \quad , \qquad (3.14)$$

where

k = intrinsic permeability, cm^2

K = permeability, cm/s

μ = dynamic viscosity, g/cm s

ρ = density, g/cm^3

g = gravitational constant, cm/s^2.

A fluid's viscosity is a measure of its resistance to flow while its density measures the degree to which gravity influences its flow behavior. For liquids other than water, these two factors influence the fluid conductivity in a porous media.

Knowing the hydraulic conductivity of a clay enables one to calculate the volumetric flow rate (Q) through a saturated, homogeneous liner composed of that clay. Given a liner area (A) of 20,000 ft^2 (about half an acre), a liner thickness of 3 ft (L), 1 ft of water over the liner making L equal 4 (1 ft water + 3 ft of saturated liner), and a permeability (K) of 10^{-7} cm/s (2.88×10^{-4} ft/d), the volumetric flow rate from Equation (3.13) will be:

$$Q = KAh/L$$

$$Q = (2.88 \times 10^{-4}\ \text{ft/d})\ (20{,}000\ \text{ft}^2)\ (4\ \text{ft}/3\ \text{ft})$$

$$= 7.68\ \text{ft}^3/\text{d}$$

$$= 57\ \text{gal/d}.$$

Decreasing the permeability by a factor of 2 will halve the flow rate.

Note that this is the flow rate after equilibrium has been reached and the liner is fully saturated. A time lapse will occur between the intrusion of the leachate into the liner and its appearance at the bottom of the liner. The topic of transit and breakthrough times is discussed in Chapter 8.

3.8.2 Hydraulic Gradient

It is necessary to conduct the tests with hydraulic gradients that are substantially greater than those encountered in the field in order to measure the permeability of compacted clay within a reasonably short time period. The two implied conditions in the Darcy equation are that the flow rate is directly proportional to the hydraulic gradient and that the relationship between flow rate and hydraulic gradient is linear through the origin. There is no single accepted hydraulic gradient for use in permeability testing. Thus, gradients of 5 to 20 are recommended by some (Zimmie, 1981) while gradients as high as 362 have been used by others (Brown and Anderson, 1982).

Over the past several decades, several studies have been aimed at evaluating the validity of Darcy's law by measuring the dependence of permeability on hydraulic gradient. Oakes (1960), Hansbro (1960), Mitchell and Younger (1967), and others have published data that indicate a departure from linearity at low hydraulic gradients. Bowles (1979) observes that in clays a threshold gradient of 2 to 4 may be necessary to produce any flow. The departure from linearity at low hydraulic gradients may not be unexpected according to Yong and Warkentin (1975). The binding forces between water molecules and clay surfaces, in effect, create immobilized hydrodynamic layers of water surrounding each clay particle. The thickness of these immobilized hydrodynamic layers depends upon the extent of interaction between the water and clay surfaces and upon the driving force for flow. At sufficiently low hydraulic gradients, the "effective" pore diameter available for flow could be decreased by the immobilized hydrodynamic layers. Several workers, however, have attributed apparent threshold gradients to experimental artifact.

The current regulations require that a landfill liner have a leachate collection system that will ensure that the leachate depth over the liner does not exceed 30 cm (1 ft). Thus, for liners exceeding 1 foot in thickness, the hydraulic gradient will be less than 2. In view of the larger hydraulic gradients used in laboratory tests, the question of the linearity of permeability with hydraulic gradient should be resolved, if meaningful translation of laboratory results to the field situation is to be expected.

3.8.3 Permeability Measurement Factors That Influence Test Results

Permeability may be determined by either a constant head test or a falling head test. In a constant head test, as implied by the name, the test sample is subjected to the permeant fluid under a constant head (i.e., the hydraulic gradient is constant). In falling head tests, the head of permeant fluid is allowed to decrease (fluid passes through the sample) during the timed interval of the test.

Equations (3.15) and (3.16) are used to calculate permeability through the methods of constant head and falling head, respectively.

For constant head, $$K = \frac{QL}{hA} \text{ , and} \tag{3.15}$$

for falling head, $$K = \frac{aL}{At} \ln \frac{h_1}{h_2} \text{ ,} \tag{3.16}$$

where

K = hydraulic conductivity, cm/s

L = length of soil path across which head is impressed, cm

Q = volumetric flow rate, cm^3/s

A = cross-sectional area of sample, cm^2

h = hydraulic head, total hood

t = time interval over which the sample is collected (or readings are taken)

a = cross-sectional area of in-flow column

h_1, h_2 = height of fluid in in-flow column at beginning of test and at the end of test.

The total head is actually comprised of three components: velocity, elevation, and pressure head. Because the seepage velocity in geotechnical problems is relatively low, the velocity head is insignificant compared to elevation and pressure heads. As used in this report, total or hydraulic head refers to the combination of the elevation difference between the in-flow and the out-flow fluid levels and any applied pressure or vacuum expressed as an equivalent height of water column.

Most permeability test devices may be operated in the laboratory in either the falling head or constant head mode, depending on the hydraulic systems connected to the permeameter. Similarly, in the field either a constant head or a falling head test may be used to determine flow through the soil. Theoretically, test results (i.e., the value of K that is calculated) should be the same whether the measurements are made in a constant head or falling head test.

Laboratory permeability tests are conducted to predict performance of a natural or compacted clay liner in the field. Frequently, however, to complete a test within a reasonable time frame, laboratory conditions are established that differ substantially from conditions in the field (e.g., elevated hydraulic gradient). An understanding of the many factors that affect permeability tests is necessary to extrapolate, in a meaningful way, from test results to performance prediction.

Important factors that influence permeability measurements include sample characteristics and preparation, permeant properties, design of the test apparatus, and selection and control of variables during test performance. Another important factor that has not been addressed adequately is interlaboratory variability in permeability tests. Regarding variability in permeability, Zimmie (1981) has concluded that, "One is always dealing with orders of magnitude in permeability problems, and it is unrealistic to expect results to agree within less than several hundred percent."

A summary compiled by Olson and Daniel (1979) of potential errors in laboratory permeability tests is given in Table 3-4. The information is based on reported permeability test results from several investigations. Another summary of sources of error in estimating field permeability from laboratory tests is given in Table 3-5. This table is reproduced from Daniel (1981).

3.8.3.1 Sample Selection, Size, and Preparation--

Measuring the permeability of compacted fill is often accomplished by bringing undisturbed samples into the laboratory. Samples obtained with Shelby tubes or similar sampling devices are considered "undisturbed," although in reality this is not the case. Specifying sample selection, sample size, sample preparation, and the number of samples subjected to testing should be made based on adequately representing field conditions. The number of samples and tests should be determined by the level of statistical confidence desired combined with the precision and accuracy of the test method employed. Confidence limits for permeability as a function of numbers of samples have been determined by Mason et al. (1957) for tests conducted in a laboratory pressure cell. Their results are shown in Table 3-6. Comparable data are not available for field measurements or for measurements made on field-compacted samples.

Large samples tested in situ (see Section 3.8.5) may be of sufficient size to include some macropores, holes, natural cracks, or sand lenses. Such samples are more likely than much smaller samples to provide good estimates of actual field performance. However, the time required for such

TABLE 3-4. SUMMARY OF POTENTIAL ERRORS IN LABORATORY PERMEABILITY TESTS ON SATURATED SOIL[a]

Source of error	Direction of deviation of measured permeability from correct value	Ratio of measured permeability to correct value
Voids formed in sample preparation	High	>1
Smear zone formed during trimming	Low	<1
Use of distilled water as a permeant	Low	0.005 to 0.1
Air in sample	Low	0.1 to 0.5
Growth of microorganisms	Low	0.001 to 0.1
Use of excessive hydraulic gradient	Low or high	<1 to 5
Use of wrong temperature	Varies	0.5 to 1.5
Ignoring volume change due to stress change	High	1 to 20
Flowing water in a direction other than the one of highest permeability	Low	1 to 40
Performing laboratory rather than in situ tests	Usually low	<0.0001 to 3

[a] Data from Olson and Daniel (1979).

TABLE 3-5. SUMMARY OF SOURCES OF ERROR IN ESTIMATING FIELD PERMEABILITY OF COMPACTED CLAY LINERS FROM LABORATORY TESTS[a]

Potential sources of error	Laboratory K too high or low?	Possible number of orders of magnitude of error[b]
Compaction at a higher water content in laboratory than in field	Low	1 to 3
Maximum size of clay chunks smaller in laboratory than in field	Low	1 to 2
Deleterious substance (e.g., plant roots) present in the field but not in laboratory samples	Low	1 to 3
Use of more compactive effort in the laboratory than in the field, resulting in optimum water content being higher in field than in laboratory	Low	1 to 3
Air in laboratory samples	Low	0 to 1
Use of excessive hydraulic gradient causing particle migration	Low	0 to 1
Lack of steady-state seepage due to stress change	High	0 to 1
Sample size too small in laboratory test	Low	0 to 3
Desiccation cracks in field	Low	No data

[a] Reproduced from Daniel (1981).

[b] A rough estimate based on available data.

TABLE 3-6. CONFIDENCE LIMITS FOR PERMEABILITY (K) AS A FUNCTION OF NUMBER OF SAMPLES AND THE MEAN PERMEABILITY[a]

Number of samples	Confidence limit, ΔK[b]
2	2.00 K
3	1.90 K
5	1.45 K
8	1.11 K
16	0.76 K

[a] Data from Mason et al. (1957).

[b] Confidence limits were computed with a variance of 0.1 (the lower limit of the variation observed in samples taken from a given pit or site) and t-distribution value appropriate to a 95-percent confidence limit.

tests may be prohibitive, particularly if the purpose of the test is construction quality control. Laboratory tests are performed on smaller samples but have the advantage of a shorter test time and limited interruption of construction activities.

During the design phase of a clay liner construction project, samples of clay soils are obtained from the site and subjected to a series of laboratory tests, which include the measurement of their permeability in the compacted state. The manner in which these samples are handled and the procedures followed during compaction can greatly influence the test outcome. At present, there are no accepted standard protocols for test sample preparation.

A recent study of test methods used by commercial soil laboratories (Truesdale et al., 1985) revealed that some used protocols requiring air drying the soil while others required that it be maintained at or near the field moisture content before clod size reduction. Reasons given for drying were that it made it easy to break up clods, sieve the soil, and obtain a very homogeneous soil mass for testing. The problem with drying, however, is that there is no way of knowing if the dried soil can be rehydrated to its former condition. Sangrey et al. (1976) found that drying and rewetting significantly altered the liquid limits of several clays. Typically, rehydration took several weeks rather than the 24 hours commonly allowed for most laboratory tests. More important was their finding that some clays were irreversibly altered by drying.

Truesdale also found a variety of methods for reducing clod size and obtaining a uniform representative sample for compaction. When dry soil was used, the clod size reduction methods included grinding the clods in an electric mill, breaking the clods up with a hammer, and manually crushing them between a hand-held steel plate and a counter top. Typically, the ground sample is passed through a No. 4 sieve before being rewet and compacted.

Wet samples were also handled in a variety of ways. These included manually breaking the clods, forcing the moist clay through a No. 4 sieve, and mixing and grinding the soil in a Hobart® vegetable shredder.

Several techniques have been developed for compacting clay soil samples. In general, the soil is compacted in a cylindrical compaction mold. If a fixed-wall permeation test is to be run, the clay can be compacted directly in the permeameter, which in this instance serves as the compaction mold. Alternatively, clay can be compacted in a separate compaction mold, extruded, and then trimmed to fit the permeameter. At some facilities, a split mold is used.

A variety of methods are available for compacting test samples in the laboratory. Among these are:

- Static Compaction--A hydraulic or mechanical press is used to compress a predetermined weight of soil into a mold of known volume (to find the required density).

- Impact Compaction--A drop hammer is used to compact the specimen. Often the compaction is done according to the ASTM procedure for determining the moisture density curve. The procedure is often modified to accommodate various-sized compaction molds.

- Kneading Compaction--This may be accomplished with a device known as a Harvard Miniature Compactor. Some investigators think this device mimics the kneading compaction obtained in the field with sheepsfoot rollers.

- Manual Compaction--In some laboratories, soil samples are weighed and pushed into a mold of known volume with a hand-held ram or rod.

Recently, Dunn and Mitchell (1984) reported on a series of permeability tests in which the only difference was the method of compacting the samples. When samples were compacted by different methods to both 90 and 95 percent of their maximum dry density, differences in their hydraulic conductivities were notable. At both dry densities, the static compaction produced samples with higher hydraulic conductivities than kneading, which was second highest, or impact, which was lowest. Manual compaction methods were not tested.

The results of Dunn and Mitchell's (1984) study indicated that static compaction produced replicate samples with the least effort. The higher hydraulic conductivity obtained with this method correlates better with field-compacted samples than the other commonly used compaction methods.

Regardless of the compaction method used in the laboratory, the final sample should not be considered equivalent to the same material compacted in the field, for several reasons. Field compaction is done with machines whose size and compactive effort are not duplicated in the laboratory. Clod sizes, which are very small and fairly well controlled in the laboratory, are much more variable in the field, where clod size may range up to 6 or 8 inches or even greater in some cases. Finally, moisture content and distribution in laboratory samples are controlled much more carefully than in the field, where the combination of large clods and inadequate curing (soaking) time after water addition may result in nonuniform moisture distribution within a liner. The profound effect of moisture content on permeability is discussed in Chapter 2 (Section 2.4.2).

The significance of sample diameter has been investigated by Daniel (1981). His data show that permeability determined on the specimens tested in the laboratory drastically underestimated the actual permeability of a clay liner. Test results on different diameter samples of a compacted clay liner are shown in Table 3-7. The average permeability of the in-place liner was back-calculated from measured leakage rates and found to be 1×10^{-5} cm/s. Boynton and Daniel (1985) compacted test samples with various diameters ranging between 15 and 6 inches. The measured hydraulic conductivities showed an increased sample diameter, with the smallest

TABLE 3-7. TEST RESULTS SHOWING EFFECT OF SAMPLE DIAMETER ON PERMEABILITY MEASUREMENTS[a]

Sample diameter (cm)	Permeability (cm/s)
3.8	1×10^{-7}
6.4	8×10^{-9}
243.8	3×10^{-5}

[a] Data from Daniel (1981).

diameter having the lowest conductivities. However, the highest and lowest conductivities differed only by a factor of 2, which the authors did not consider to be of practical significance.

Anderson and Bouma (1973) experimented with a series of cores of different lengths to determine the effect of sample size on permeability. They found that permeabilities for cores 17 cm in length were lower by one-half an order of magnitude than for the 5-cm-length samples.

Whether a sample is compacted dry of optimum or wet of optimum can profoundly influence the resulting permeability. Daniel (1981) reported that the permeabilities of soils compacted dry of optimum might typically be 10 to 1,000 times greater than permeabilities of the same soils compacted wet of optimum (this phenomenon is discussed in Section 2.3). Because moisture conditions at compaction can strongly influence permeability measurements, gross errors in predicting field permeability from laboratory tests may occur if compaction is performed at different water contents. Permeability tests can be conducted with samples compacted at several points in a range of moisture contents to ensure that actual field moisture conditions are reproduced.

Another consideration is that equipment used to compact laboratory samples does not resemble field compaction equipment. Lambe and Whitman (1979), in comparing field and laboratory compaction efforts, have concluded that generally the laboratory curves yield a somewhat lower optimum water content than the actual field optimum.

In the field, a compacted low-permeability soil may display anisotropic flow characteristics. The horizontal permeability (parallel to the plane of compaction) of anisotropic soil may be much higher than the vertical permeability (normal to the plane of compaction). Such a condition will lead to predominantly horizontal flow. For in-place stratified clays, the ratio of horizontal to vertical permeabilities may exceed 10 (Olson and Daniel, 1979).

In contrast, however, Boynton and Daniel (1985) report that no anisotropic flow was found in laboratory-prepared samples of fireclay compacted wet of optimum. Anisotropic flow found in compacted soils may be the result of poor lift bonding or variations in density within the test sample.

3.8.3.2 Hydraulic Gradient--

Since Darcy's law indicates a linear relationship between flow rate and hydraulic gradient, many workers have used elevated hydraulic gradients to reduce testing time. However, if hydraulic gradients are excessive, piping (opening flow channels and increasing hydraulic conductivity) or particle migration (blocking flow channels and reducing hydraulic conductivity) may occur and can significantly influence permeability measurement. Although such effects can occur and have been reported (Daniel, 1981; Mitchell and Younger, 1967), studies have been conducted at elevated gradients with no evidence of piping or particle migration (Anderson, 1981).

Excessive hydraulic gradients can result in deviations from Darcy's law, which is only valid for laminar flow conditions. Even under gradients as high as 361, however, velocities low enough to assure a Reynolds number within the laminar flow regime are assured due to the very small particle diameters in fine-grained soils.

Zimmie et al. (1981) have recommended use of hydraulic gradients between 5 and 20 for laboratory studies. Research performed at Louisiana State University has lead to the conclusion that tests conducted under hydraulic gradients as high as 100 are "acceptable for testing, reducing testing times to realistic and practical duration" (Acar and Field, 1982). Dunn and Mitchell (1984) reported that increasing the gradient in steps from 20 to 200 caused an irreversible decrease in hydraulic conductivity. They attributed most of the observed changes to particle migration due to seepage forces. They recommend that test gradients be kept as low as possible while still allowing the test to be completed in a reasonable time.

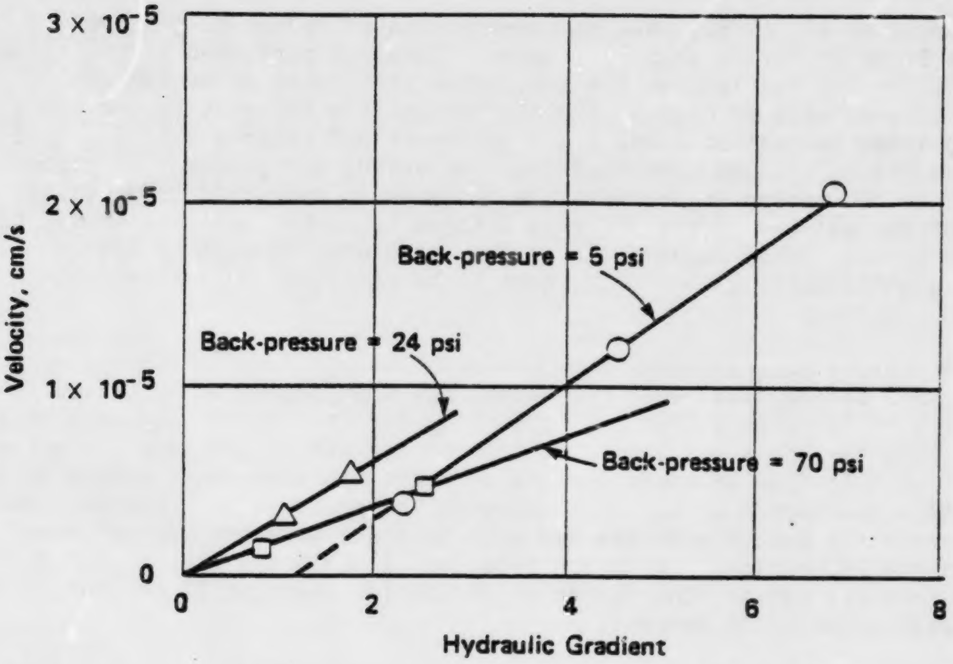
3.8.3.3 Sample Saturation--

A soil sample, even when compacted, has some degree of porosity. The pores are filled with either gas (generally air) or liquid. Because water cannot flow through a gas bubble, entrapped air within the interconnected pores that form flow channels for the permeant fluid causes a reduction in flow and a corresponding apparent decrease in permeability. Soaking the sample from the bottom with the top open to the atmosphere may not result in complete saturation. Smith and Browning (1942) found that in 200 specimens soaked from the bottom, the degree of saturation averaged 91 percent, with the lowest value at 78 percent.

The extent of the error in permeability measurement attributable to entrapped gas bubbles is not fully known, although decreases in permeability by factors ranging from 2 to 5 have been reported (Johnson, 1954).

The use of backpressure in permeability testing is considered by some researchers to be necessary for saturating a soil sample. Matyas (1967), investigating the effect of saturation on permeability, applied a backpressure of 5 psi to "saturate a clay specimen." According to his calculations, this pressure was not sufficient to guarantee complete saturation. After the saturation attempt at the low pressure, the specimen was subjected to different hydraulic gradients, and discharge velocities were measured. Similar samples were saturated at 24 psi and at 70 psi. The plot of hydraulic gradient versus discharge velocity is shown in Figure 3-16.

Matyas' data indicate that at 5 psi backpressure the velocity gradient plot does not pass through the origin and is nonlinear. However, when the backpressure was raised to either 24 psi or 70 psi, the velocity gradient relationship was found to be linear and to pass through the origin. The apparent nonlinear velocity gradient relationship was attributed to the presence of air in the voids. There was no evidence of a threshold gradient (i.e., a gradient below which flow would not occur).



After Matyas, 1967

Figure 3-16. Effect of backpressure on permeability to water, Sasumua clay.

Matyas (1967) did not address the difference in the velocity gradient relationships at backpressures of 24 and 70 psi. One might conclude that the application of higher backpressures caused some particle rearrangement resulting in lower observed permeability values. The lowest practical backpressure to obtain saturation appears desirable in order to minimize these effects. The effects can also be minimized if the backpressures are increased incrementally and sufficient time is allowed between successive increases so that high seepage forces do not develop in unsaturated portions of the specimen. Daniel et al. (1984) reports using backpressures of 40 to 60 psi applied in 10-psi increments. Each increment is held for a period ranging from several minutes to several hours. Full backpressure is maintained for 1 to 5 days before the test is begun.

3.8.3.4 Permeant Characteristics--

Permeant fluids that are commonly used to determine baseline permeability values include deionized water, tap water, groundwater (representative of a specific site) and standard permeant solution, typically 0.01 N calcium sulfate or calcium chloride. The standard solution is intended to simulate the hard water frequently used in the field during installation and subsequent hydration of a clay barrier.

While some researchers claim that use of different fluids can influence baseline permeabilities, others do not believe the effects are significant (Olson and Daniel, 1981). Recently, the trend has been toward using the standard calcium solutions for initial saturation and baseline permeability determinations.

In quality control and in compatibility testing, factors that influence the flow characteristics of a permeant fluid will alter the apparent soil permeability to that fluid. Because viscosity and density are temperature-dependent properties of a fluid, seasonal temperature changes in the field could influence the range of permeabilities to be anticipated. Large variations in temperature over a test sample are unlikely, however. Soil permeabilities are sometimes expressed as intrinsic permeabilities (see Equation 3.15 in Section 3.8.1) to account for differences in flow properties between different permeant fluids, or at different temperatures. However, the use of the permeability coefficient, K, is much more common and is used throughout the remainder of this section.

The presence of certain chemicals in permeant fluids has been shown to alter the permeability of clay soils (see Chapter 4). Shrinkage of clay particles may result from changes in interlayer spacing due to interaction of chemicals with the clay particle's double layer. Changes in adsorbed cations or the presence of salts or acids in the permeant fluid can also effect significant permeability increases. Such effects are the reason for conducting compatibility tests. Effects of chemical permeants may be evident immediately after the permeant penetrates the sample, or the effect may not be seen until one or more pore volumes have passed through the sample. These effects are discussed fully in Chapter 4.

3.8.3.5 Test Duration--

A number of factors can cause changes in permeability with time. It is essential in permeability testing that flow through the sample be continued until stable permeability measurements are obtained. In compatibility testing, several pore volumes of fluid should be passed through the sample to ensure that any tendency toward an increased or decreased permeability is observed. A final permeability cannot be established if the permeability changes appreciably with time.

Changes in pore pressure cause changes in sample volume. In a constant head test, some of the initial measured in-flow into the sample compensates for the volume change rather than being the result of steady-state seepage (Olson and Daniel, 1979). Permeability tests should be run long enough to ensure that steady-state values are obtained.

Some clays exhibit thixotropic behavior; their internal structure and flow characteristics change with time. These changes can increase permeability with time because of larger effective pore sizes in the clay. Mitchell et al. (1965) found measured permeabilities to be as much as six times greater for samples tested at 21 days compared to samples tested immediately after compaction. Dunn and Mitchell (1984) reported similar results. Boynton and Daniel (1985) did not observe thixotropic changes in stored samples; but they attributed this to their having compacted them dry of optimum, whereas Mitchell et al. (1965) and Dunn and Mitchell (1984) compacted wet of optimum.

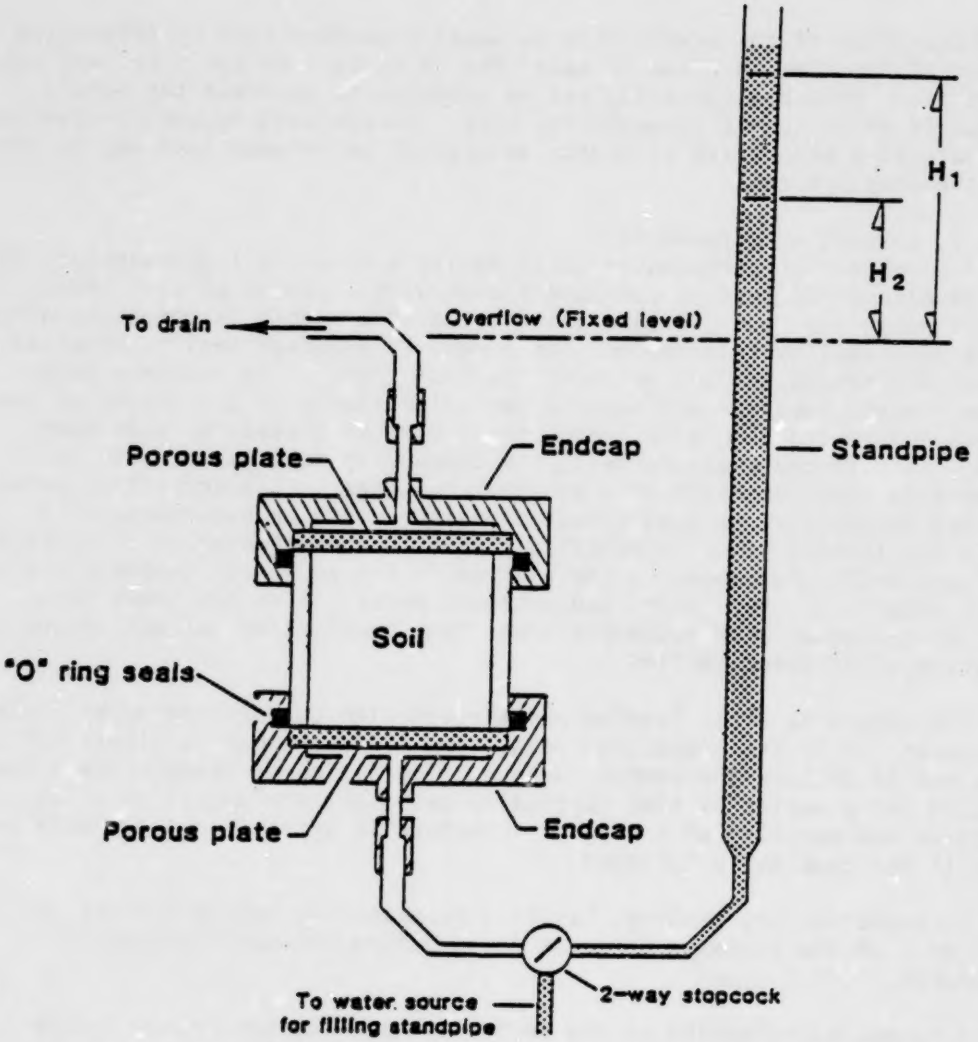
Although not usually a significant factor, microorganisms present in test samples have been shown to influence permeability by clogging the flow channels with organic matter or with gases produced by the microorganisms. Allison (1947) reported long-term permeabilities of sterile soils to be 8 to 50 times lower than the values of nonsterile soils.

3.8.4 Laboratory Permeability Tests

Examples of permeability test devices and the methods used for testing are described briefly below. The emphasis of the discussion is on features that distinguish the various devices and their advantages and disadvantages.

3.8.4.1 Pressure Cell--

The accepted standard test for laboratory determination of saturated permeability in the agricultural sciences (but not the geotechnical sciences) is called the pressure cell test. In this procedure a soil sample, which may be an undisturbed core, a sample compacted in a mold, or a volume of soil, is placed in a metal pressure cell. After the soil is initially saturated, it is connected to a standpipe. The permeant fluid is introduced through the standpipe and forced through the pressure cell under a falling head. The standpipe may or may not be connected to a source of air pressure to superimpose a pressure head over the fluid column. The pressure cell apparatus for a falling head test is illustrated in Figure 3-17. A constant head test could also be conducted in the pressure cell by measuring the volume of the effluent forced through the sample within a timed interval.



After Klute, 1965

Figure 3-17. Apparatus for pressure cell method.

Saturation of the sample core is usually accomplished by submerging one end of the core in a pan of water for 16 hours with the other end open to the air. This procedure may not be adequate to saturate the sample completely prior to the permeability test. Vacuum wetting and fluctuating external gas pressure are alternate saturation techniques that may be used with the pressure cell.

3.8.4.2 Compaction Permeameter--

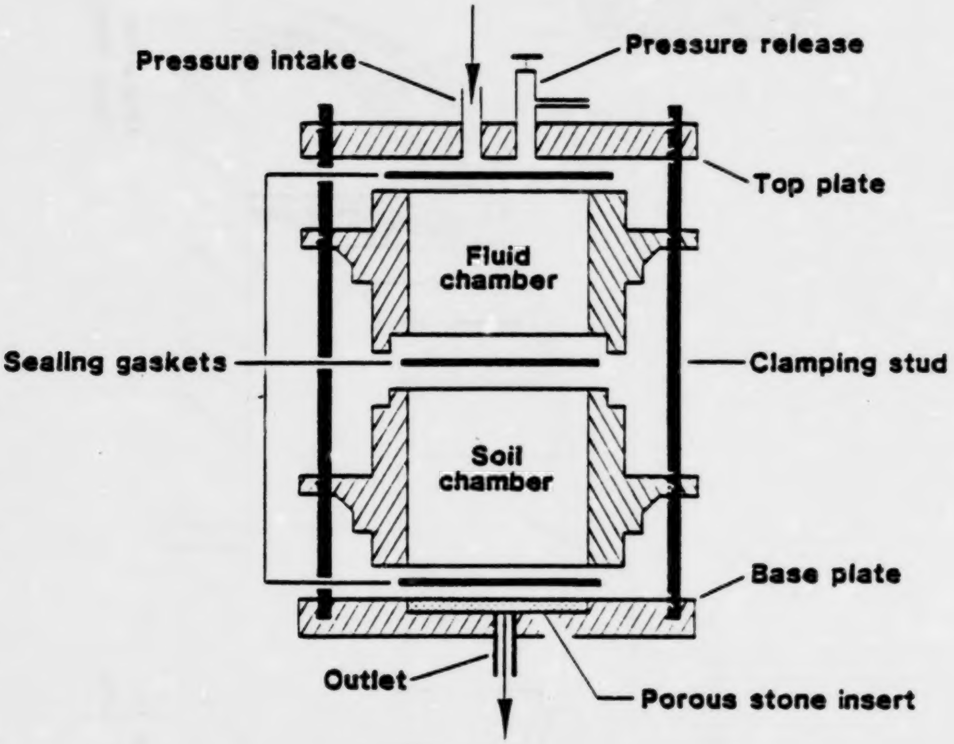
The compaction permeameter (also called a fixed-wall permeameter) has been developed for testing permeant fluids with a compacted soil layer. One advantage over the pressure cell is that the sample is compacted directly in the permeability test device. As a result, a better seal is obtained between the sample and the walls of the test vessel. To ensure a good sidewall seal, some workers apply a bentonite slurry to the inside of the chamber before the sample is introduced. Silicon grease has also been tried. As with the pressure cell, the compaction mold may be used in either a falling head test or a constant head test. The compaction permeameter may be modified so that elevated pressures can be superimposed to reduce the testing time. A modified compaction permeameter is illustrated in Figure 3-18. Not shown on the diagram is the source of compressed air with a water trap, regulator, and pressure meter. Also not shown is a fraction collector with automatic timer that handles the collection and isolation of effluent samples.

The sample is first leveled and sprayed with standard permeant fluid to saturate it in the compaction mold. The fluid chamber is slowly filled so as not to disturb the sample surface. The filled permeameter is allowed to stand for a period of time (typically overnight or longer) to allow hydration and swelling of the clay. Pressure is applied to the sample only after it has completely hydrated.

In compatibility testing, baseline permeability determinations are obtained with the standard permeant fluid before the test permeant is introduced.

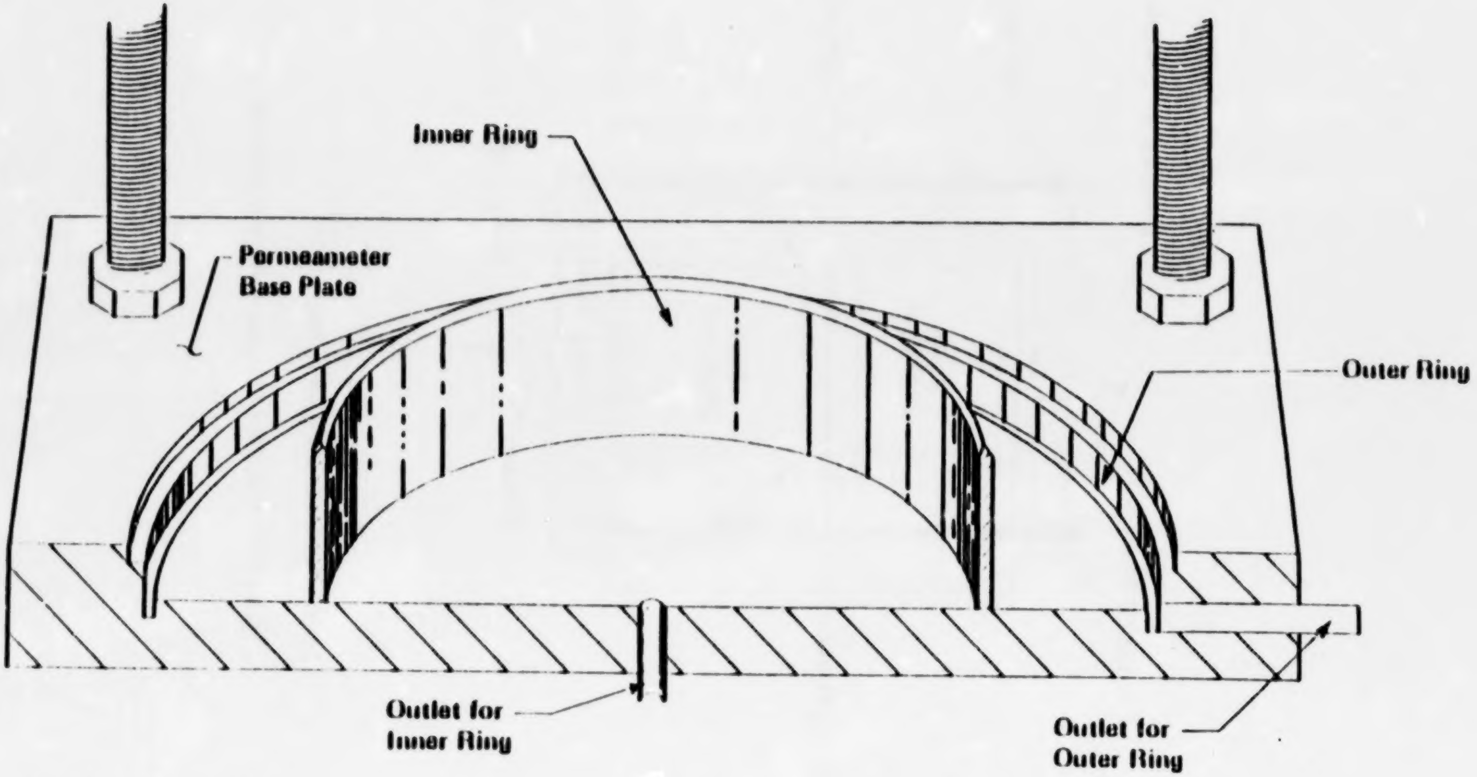
A recent modification of the compaction permeameter device is the double-ring permeameter. This apparatus has been developed for use in compatibility tests involving waste leachates that may cause sample volume changes during testing (Anderson, 1983). In the double-ring permeameter test, a soil sample is compacted in a cylindrical mold that is mounted on a base plate with a short cylindrical section mounted on its upper surface (Figure 3-19). Separate outlets in the inner and outer rings serve to separate sidewall leakage from flow through the central soil matrix. This type of device may be especially useful for tests with soils that are liable to undergo volume changes during permeability testing. Also, the double-ring permeameter is appropriate for use in compatibility tests that involve liquids that may cause shrinkage of the sample due to development of soil structure.

Testing in the double-ring permeameter begins by passing sufficient standard permeant fluid through the soil to obtain stable baseline perme-



Source: Anderson and Brown, 1981

Figure 3-18. Modified compaction permeameter.



Source: Anderson, 1983

Figure 3-19. Detail of the base plate for a double-ring permeameter.

abilities for both the inner and outer compartments. If the permeability of the outer compartment is significantly higher than that of the inner compartment, the soil core is discarded and a new core is compacted in the mold. After stable baseline permeabilities are obtained, the standard permeant is replaced by the test fluid, which is then passed through the sample. The volume passed through each compartment is measured and reported based on the pore volume of that compartment.

Tests in the double-ring permeameter are usually conducted with a constant elevated hydraulic gradient. They also can be performed as a constant head test without superimposed air pressure. As in other tests, 100-percent saturation of the sample may not be achieved unless backpressure is used.

Permeability value, leachate volume, and time increment are recorded for discrete volumes of permeant fluid passed through both chambers during a compatibility test. Permeability measurements are then plotted versus the total number of pore volumes passed through the sample. Data plotted in this way indicate changes in permeability that might be expected for a clay liner in service over a period of time that corresponds to the number of pore volumes exchanged in the test.

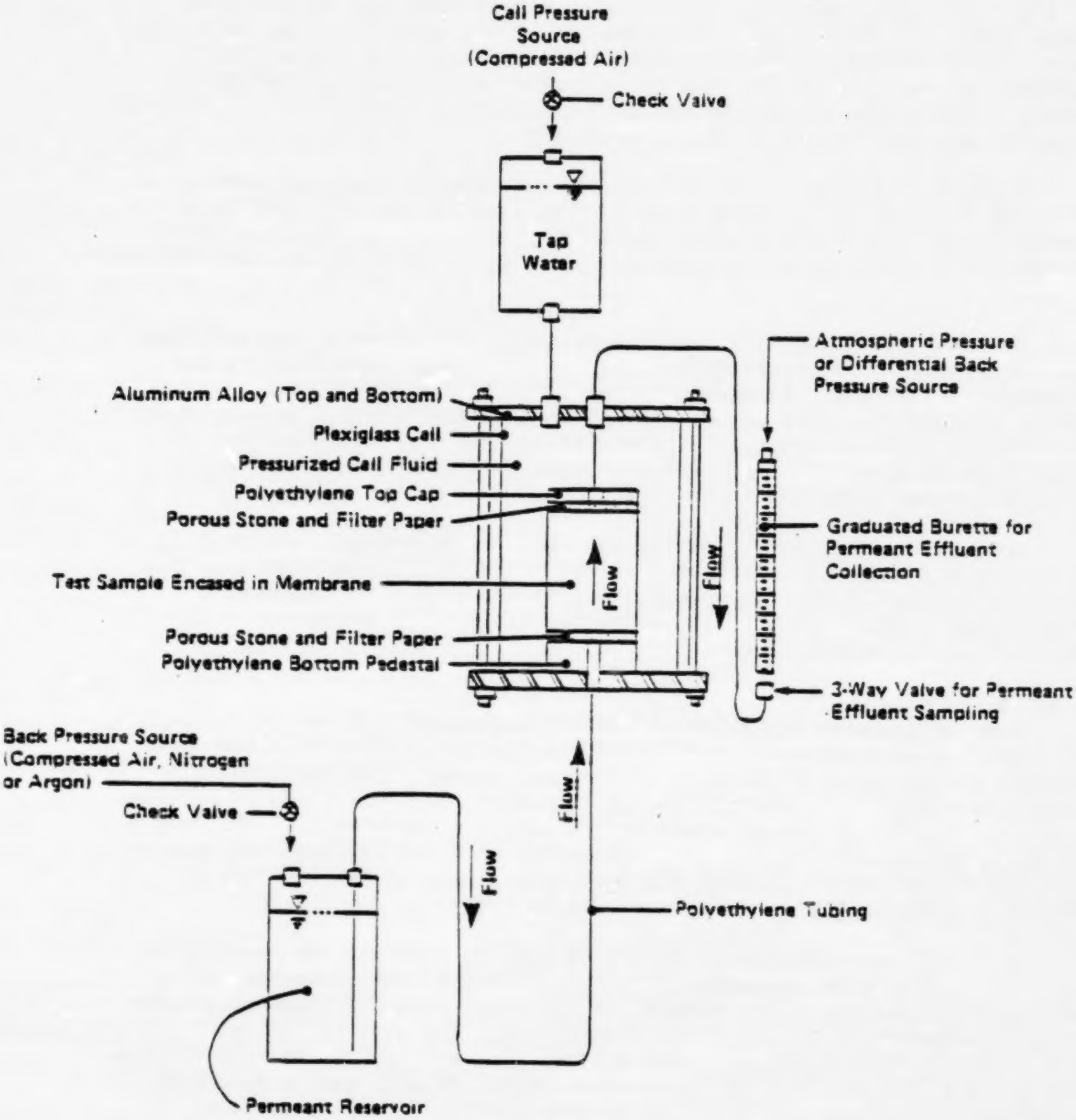
Because the double-ring device has been introduced only recently, a criterion has not been established to determine what constitutes a significant difference between the permeability in the central compartment and that in the annular space. This criterion needs to be developed if the device is to be widely used.

3.8.4.3 Triaxial Cells--

Samples prepared for triaxial testing consist of cylindrical columns of compacted soil encased laterally with a flexible membrane (often latex rubber) and enclosed at the ends with porous stones. The enclosed soil sample is placed in a water-filled cell that can be pressurized to provide a confining pressure on the sides of the sample (Figure 3-20). The sample to be tested may be prepared in a compaction mold and extruded for testing in the triaxial cell. Samples are typically 2 to 3 inches in diameter. Undisturbed Shelby tube samples may also be tested.

Permeability tests are performed by passing permeant liquids upwards into the sample under pressure while maintaining a lower pressure at the exit port on the top of the sample. A confining pressure somewhat greater than the pressure under which the liquids enter the soil is imposed to press the flexible membrane firmly against the soil sample, preventing flow along the sidewall. The confining pressure on the soil sample should be selected to simulate the lateral pressures a material will experience in the clay liner.

Sample saturation is accomplished by forcing a standard permeant liquid upward through the sample. Backpressure may be applied to hasten the dissolution of trapped air bubbles or gases generated by reactions between permeant liquid and the sample. The use of backpressure ensures


Cell Pressure Source (Compressed Air)
Check Valve
Tap Water
Atmospheric Pressure or Differential Back Pressure Source
Aluminum Alloy (Top and Bottom)
Plexiglass Cell
Pressurized Cell Fluid
Polyethylene Top Cap
Porous Stone and Filter Paper
Graduated Burette for Permeant Effluent Collection
Test Sample Encased in Membrane
Flow
Flow
Porous Stone and Filter Paper
Polyethylene Bottom Pedestal
3-Way Valve for Permeant Effluent Sampling
Back Pressure Source (Compressed Air, Nitrogen or Argon)
Check Valve
Flow
Flow
Polyethylene Tubing
Permeant Reservoir

Source: Haji-Djafari and Wright, 1982

Figure 3-20. Schematic of a constant head triaxial cell permeameter.

complete saturation of the sample, a factor that probably contributes to the good reproducibility of triaxial tests. Pressure regulators and electronic pressure transducers are used to monitor sample stress conditions and to assess the saturation state within the sample during testing.

Sufficient standard liquid is passed through the sample to establish a stable baseline permeability. The test permeant fluid is then introduced. The test may be conducted as a constant head or falling head test. Permeability value, leachate volume, and time increment are reported for each volume of test fluid passed through the sample. These data are used to plot permeability versus pore volumes of test fluid passed.

Excellent precision (±20 percent) based on four samples has been reported for the apparatus shown in Figure 3-20 (Haji-Djafari and Wright, 1982).

Questions have been raised about whether triaxial cells are appropriate for compatibility testing of permeant fluids that may cause the soil sample to shrink (Chapter 4). Because of the confining cell pressure, the flexible membrane will contract with the soil sample so that shrinkage cracks that could occur in a field situation and that might appear in fixed-wall devices as sidewall leaks would not be simulated in the triaxial cell. Not all researchers agree on this issue since the effect has not been demonstrated clearly in comparative tests. Boynton and Daniel (1984) have shown that cracks in desiccated and rewet samples would only partially close unless substantial effective stress were applied.

Although conventional triaxial cells are not designed to withstand concentrated chemical permeants, several modifications are possible to minimize the problem of chemical attack on the test device. Polyethylene, nylon, or Teflon® fittings may be used in place of the customary brass or steel fittings. The use of colostomy bags (of the type used in surgical applications) in place of the conventional latex membrane has also been reported for use with wastes such as gasoline (J. Withiam, D'Appolonia Consulting Engineers, Inc., Pittsburgh, PA, personal communication, 1983). Samples may also be wrapped in thin Teflon® tape to prevent contact between the permeant and the latex membrane.

3.8.4.4 Consolidation Cells--

Consolidation cells (consolidometers) are commonly used in the field of geotechnical engineering to determine the compressibility and rate of settlement of soils. Consolidation occurs when water is squeezed out of the soil and is therefore a function of permeability. A fixed-ring consolidation cell can be used to measure permeability (Figure 3-21).

The consolidation cell method is routinely used in testing permeability for applications such as earth dams, retaining walls, and slurry trenches. The method has not been widely used in the evaluation of chemical compatibility with clay liner material.

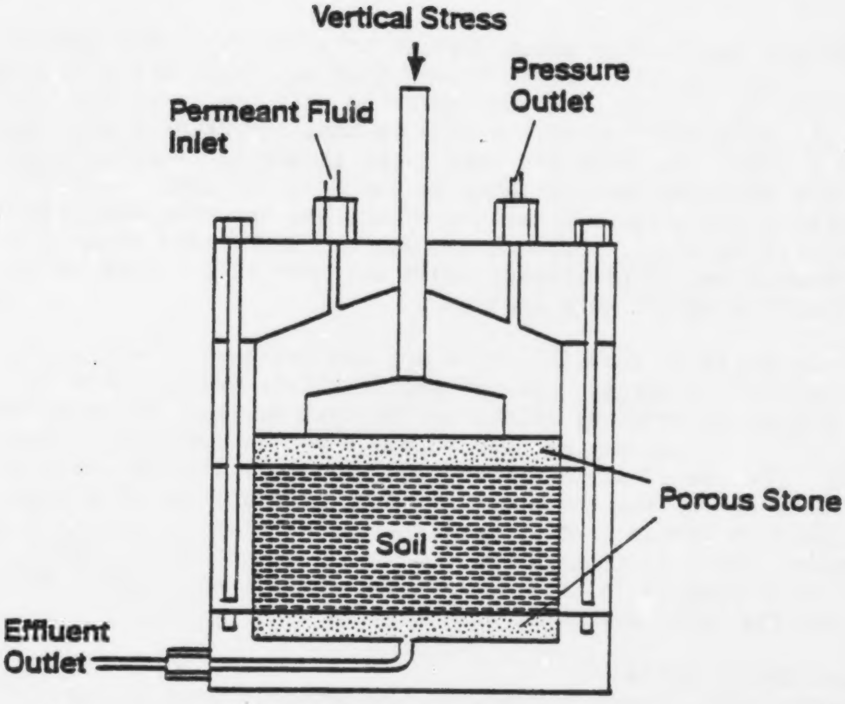


Figure 3-21. Consolidation permeameter.

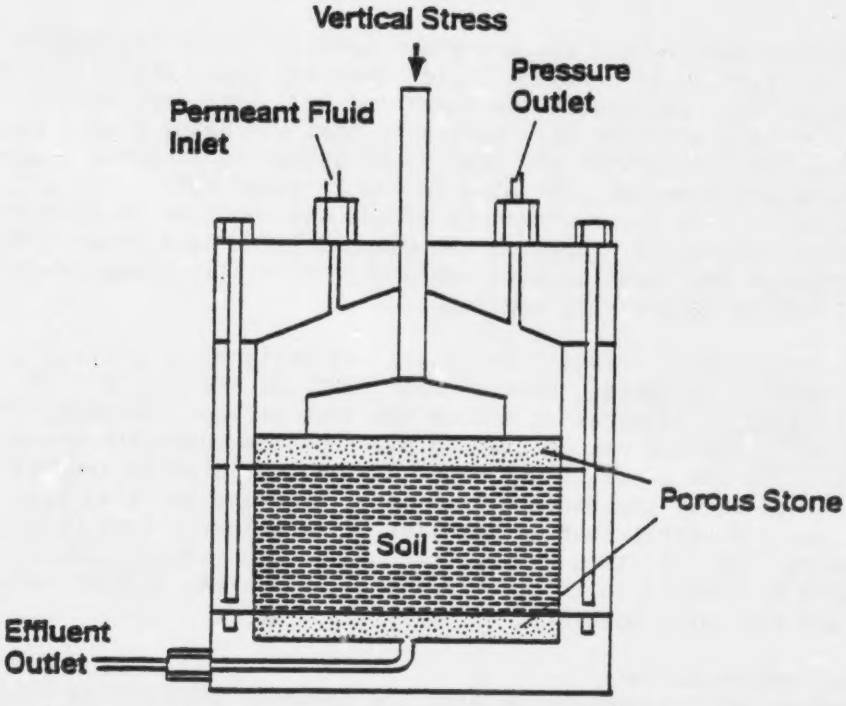


Figure 3-21. Consolidation permeameter.

3.8.5 Field Permeability Tests

The methods described below for measuring permeability in the field are frequently modified or adapted for specific sites or for specific types of investigations.

3.8.5.1 Double-Ring Infiltrometer/Permeameter--

In the double-ring infiltrometer technique (ASTM-D3385), one open metal cylinder is placed inside another, and both are driven into the soil. The cylinders are then partially filled with water that is maintained at a constant level in both rings. The amount of water added to maintain the constant water level in the inner ring is the measure of the volume of water that permeates the soil.

The metal cylinders may range from a few centimeters to a meter in diameter. A piece of folded burlap or other suitable material is used to protect the soil surface from puddling when water is first added to the cylinders. The double-ring infiltrometer is illustrated in Figure 3-22.

Information on the accuracy of the method for fine-textured soils is not available. However, good reproducibility and precision are indicated by results of using double-ring tests at a low-level radioactive waste disposal site (Luxmoore et al., 1981).

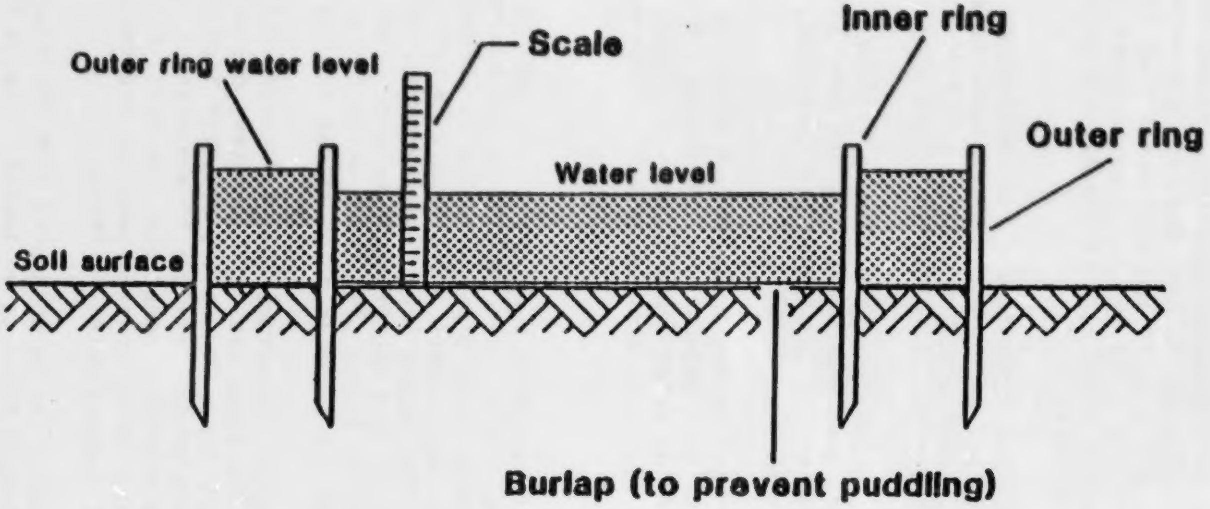
Care must be exercised in the installation of the cylinders to prevent shatter or compaction of soil adjacent to the border. Air that may become trapped below the advancing water front is another factor that can significantly affect results. In long-term tests, corrections for evaporation are needed.

3.8.5.2 Drum Tests--

A practical test to estimate permeability in the field may be conducted with two 55-gallon steel drums. The top and bottom are removed from one of the drums, forming a large steel pipe. The top is removed from the second drum, but the bottom is kept intact. The drum with both ends removed is forced into the soil. The second drum is placed close to the hollow drum, and both drums are filled to a specified level with water. The drum penetrating the soil should be filled with fresh water for at least 24 hours before any measurements are taken to reach stable saturation conditions.

After the test begins, measurements are recorded daily for 1 week or longer if necessary. The drums are refilled every day to maintain the water level close to the top. The amount of water that is lost from the intact drum is a measure of the loss due to evaporation. The additional water lost through the drum that penetrates the soil is the seepage volume. The permeability may then be calculated based on the seepage volume and the head used for the test (approximately 4 feet).

Compatibility testing can be conducted with the drum test by replacing the water with the permeant fluid to be tested. Stable permeability measurements with water should be obtained before the test fluid is introduced.



Source: EPA/Army Corps of Engineers/USDA, 1977

Figure 3-22. Apparatus set-up for double-ring infiltrometer.

The procedure described above is based on recommendations of the Federal Bentonite Company for testing a bentonite seal designed for lagoons or landfills. The specifications emphasize that the drum tests "are not of sufficient accuracy to allow for precise permeability estimates" (Federal Bentonite, 1982).

3.8.5.3 Modified Air-Entry Permeameter--

The air-entry permeameter technique tests the permeability of a 25-cm-diameter cylindrical unit of undisturbed soil that is isolated by an infiltration cylinder driven into the soil. A head of water is applied and the infiltration rate is measured until the wetting front is close to the bottom of the cylinder. This is determined with an implanted tensiometer. The basic equipment required and the experimental setup for air-entry permeameter tests are shown in Figure 3-23 (U.S. EPA, 1984).

The saturated permeability is calculated as a function of the air entry value of soil. Equations (3.17) and (3.18) are used:

$$P_a = P_{min} + G + L \quad , \qquad (3.17)$$

where

P_a = air entry value of soil, expressed as pressure head in cm water at point of air entry

P_{min} = minimum pressure head, cm water as determined by the maximum reading on the vacuum gauge

G = height of gauge above soil surface, cm

L = depth of wet front (depth of tensiometer), cm.

Saturated permeability is then calculated from:

$$K = 2 \frac{dH}{dt} \; L \frac{R_r^2}{R_c^2} /[H_t + L + 1/2(P_a)] \quad , \qquad (3.18)$$

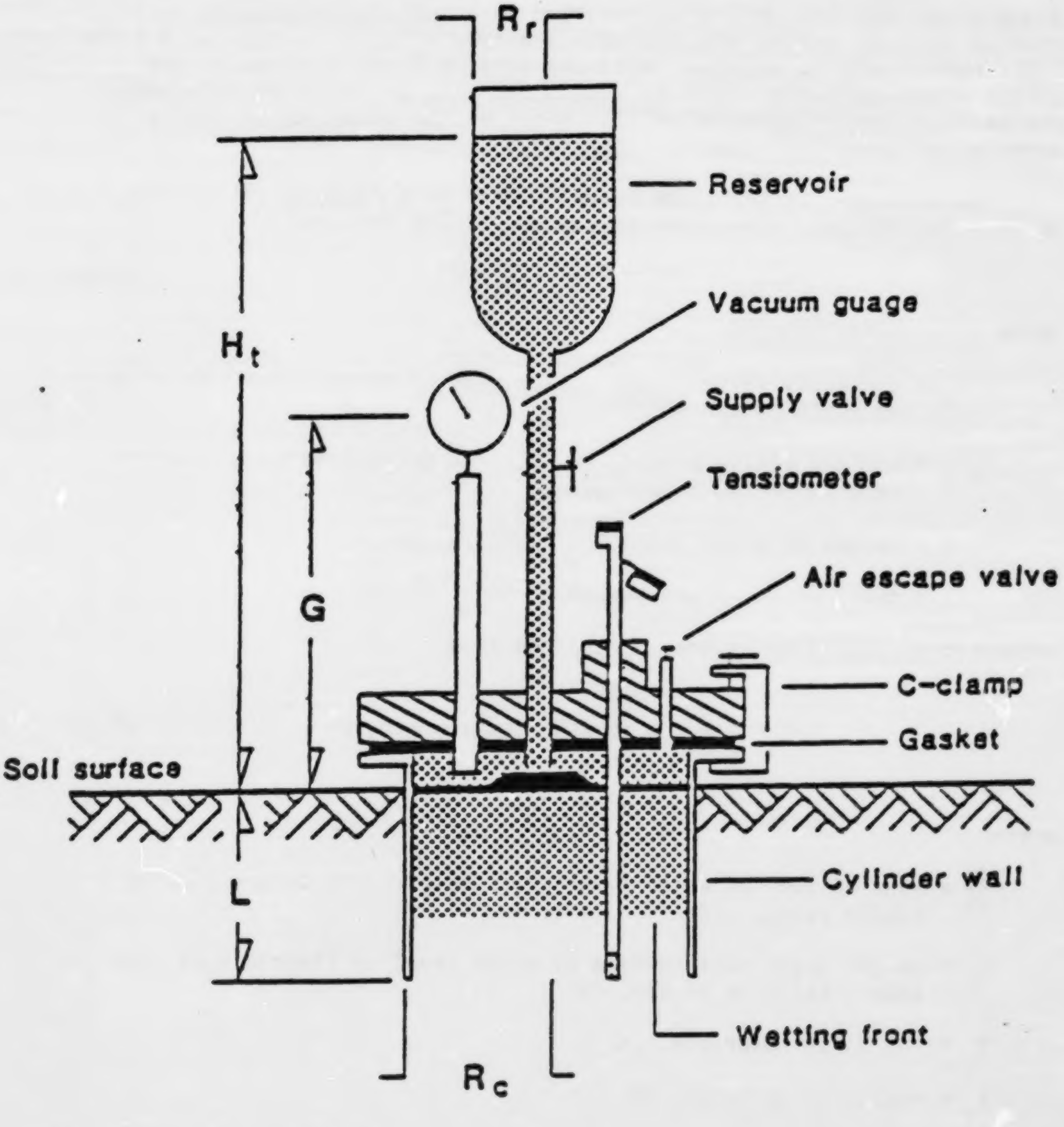
where

$\frac{dH}{dt}$ = rate of fall of water level in reservoir just before closing supply valve, cm/s

H_t = height above soil surface of water level in reservoir at time supply valve is closed, cm

R_r = radius of reservoir, cm

R_c = radius of cylinder, cm.



After U.S. EPA, 1984

Figure 3-23. Modified air-entry permeameter.

Topp and Bins (1976) found that the air-entry permeameter gave reproducible values of permeability at various depths in a number of different soils. They also found that results were consistent with values obtained from determinations made on laboratory cores. Although the laboratory core permeability was generally lower than the permeability determined by the air-entry permeameter, this was probably due to lack of worm holes or cracks in the laboratory samples. Aldabagh and Beer (1971) found the method to be consistent and reasonable with variability substantially less than the natural variation of soil.

As with other field methods, care must be exercised in the installation of the cylinder to prevent shatter or compaction of soil adjacent to the cylinder. The method is not appropriate for use on initially wet or nearly saturated soils because the induced wetting from the addition of water would not be well defined.

An advantage of the modified air-entry permeameter method over other methods is the short time period (approximately 1 hour) that is required to conduct the test. Although useful for quality control, the method is probably not applicable to compatibility testing.

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CHAPTER 4

CLAY-CHEMICAL INTERACTIONS AND SOIL PERMEABILITY

It has been known for many years that the permeability of clay soils may be drastically altered by chemicals present in the permeating liquid. Apart from the many recent studies stemming from concern over the effects of hazardous waste and waste leachates on clay liners, much research has been carried out over the last several decades to determine the effects of various chemicals on agricultural soils or on geological formations important for oil production. The potential effects of certain organic fluids on clay permeability were recognized as early as 1942, when Macey's experiments with fireclay showed that the rates of flow for certain organic liquids through clay were "of an enormously higher order than for water" (Macey, 1942). Since Macey's experiments, many researchers have investigated the effects of organic and inorganic fluids on clays in an effort to elucidate the causes of the observed changes in permeability.

The current state of the knowledge in this area is complicated by the dilemma of how to measure the permeability changes that appear to be caused by clay-chemical interactions. The question of what types of permeameters give valid measures of clay-chemical compatibility remains an important issue. Because permeability studies with different fluids have been carried out with different test protocols, different test devices, and different clays, quantitative data comparisons cannot be made except in a few of the more recent studies. Much can be learned from a review of the research that has been carried out, however, and qualitative statements can be made regarding the behavior of certain clays in the presence of many types of fluids. Unifying theories of soil physics that explain the reported findings have been advanced by several researchers. Such theories are useful for predicting clay-chemical incompatibilities that could lead to performance failures in clay-lined hazardous waste disposal facilities.

This chapter presents the experimental findings that pertain to the effects of chemicals on clay barrier permeability as well as the theories and mechanisms proposed to explain the observed effects. Section 4.1 defines the terms most important in clay-chemical compatibility testing--permeability (or, if the permeating liquid is water, hydraulic conductivity) and intrinsic permeability. Section 4.2 is a discussion of the clay-chemical interactions that influence soil permeability. A summary of the relevant permeability studies is presented in Section 4.3. Approaches that have been used in studies to determine clay-chemical compatibility and problems associated with different test methods are addressed in Section 4.4. Section 4.5 reviews in more detail permeability testing efforts that have been carried out to measure clay-chemical interactions.

4.1 PARAMETERS DETERMINED IN PERMEABILITY TESTING FOR COMPATIBILITY

When results of clay-chemical compatibility tests are compared, it is important to understand the parameters that are being measured and calculated. The parameter usually determined in compatibility tests is permeability, K, which is defined by Darcy's Law as expressed in Equation (4.1):

$$K = Q/Ai \qquad (4.1)$$

where

Q = volumetric flow rate (L^3/t)

A = cross-sectional area of flow (L^2)

i = hydraulic gradient (dimensionless).

K has units of length per unit time (e.g., cm/s). The density and viscosity of the permeating liquid as well as the pore size distribution within the soil matrix will influence the value of K. Hydraulic conductivity refers to the value of K when the permeating fluid is water. Darcy's Law is limited to saturated soil conditions and to laminar flow conditions.

In order to separate the effects of the liquid properties (viscosity and density) from those of the medium (pore size distribution), a different parameter, the intrinsic permeability, should be used. Intrinsic permeability, usually referred to as k, is a property of the medium that is dependent on the shape, size, and continuity of the pore spaces. Intrinsic permeability is "a measure of the relative ease with which a porous medium can transmit a liquid under a potential gradient. It is a property of the medium alone and is independent of the nature of the liquid and of the force field causing movement" (Lohman, 1972).

Intrinsic permeability, k, has units of length squared (e.g., cm²) and is related to permeability, K, by Equation (4.2):

$$K = k \frac{\rho}{\mu} g \qquad \text{or} \qquad k = K \frac{\mu}{\rho g}, \qquad (4.2)$$

where

ρ = density of the fluid (M/L^3)

μ = dynamic viscosity of the fluid (M/Lt)

g = acceleration due to gravity (L/t^2).

In clay-chemical compatibility testing, the value of K for a clay soil permeated by a certain chemical is determined from measurements of the fluid inflow or outflow. A change or lack of change in the value of K (when compared to K for water or other baseline fluid) may be due to a combination of two factors--

- Difference in the permeant fluid viscosity and density (compared to baseline permeant fluid)

- Change in porous medium characteristics as a result of clay-chemical interactions.

In order to separate these effects, it is usually necessary to report the results of the tests in terms of intrinsic permeability (k) both for the tests with the baseline permeant fluid and for the tests with the chemical permeant fluid in question. In practice, most researchers report and discuss their test results in terms of permeability (K) rather than intrinsic permeability (k). Provided the density and viscosity of the test fluid (at the test temperature) are known, one could calculate k to correspond to each K value reported. In general, only substantial permeability changes are meaningful in clay-chemical compatibility testing. When large changes in permeability are measured during the course of a test, the clay-chemical interaction is apparent regardless of whether the k values are computed and plotted.

4.2 CLAY-CHEMICAL INTERACTIONS THAT INFLUENCE PERMEABILITY

Mechanisms whereby the chemical nature of a permeant fluid may alter clay soil permeability and theories to predict clay-chemical interactions have been described by several researchers, among them Mitchell (1976), Brown and Anderson (1980), Acar and Field (1982), Evans, Chaney, and Fang (1981), Anderson and Jones (1983), Daniel (1982, 1983, 1984), Dunn (1983), Monserrate (1982), Peirce (1984), and Griffin and Roy (1985). In addition to laboratory and field permeability tests, methods such as X-ray diffraction, shrink-swell measurements, settling tests, and other techniques have been used to investigate the clay-chemical interactions that influence permeability.

Changes in the permeability of clay soils due to chemical interactions may result from--

- Alterations in soil fabric stemming from chemical influences on the diffuse double layer surrounding clay particles

- Dissolution of soil constituents by strong acids or bases

- Precipitation of solids in soil pores

- Soil pore blockage due to the growth of microorganisms.

The permeability of a soil may also be affected by the pore fluid velocity; high velocities can displace small particles in the soil matrix. The fluid flow velocity can also influence chemical interactions that depend on the time of contact between the soil and some chemical component of the permeating fluid. Thus, permeability and fluid flow velocity are interrelated characteristics of a soil-permeant fluid system.

4.2.1 Soil Fabric and Permeability

The permeability of any soil depends upon the geometric characteristics of the area available for fluid flow. Influencing factors are the size, shape, tortuosity, and degree of interconnection between the pore spaces. The geometric arrangement of the soil particles will determine these pore space characteristics.

The term "fabric" refers to the arrangement of particles, particle groups, and pore spaces in a soil. Modes of particle association orientation in clay suspensions were described by van Olphen (1963) as "dispersed," "aggregated," "flocculated," or "deflocculated" (see discussion in Section 2). Particle associations corresponding to these descriptors are illustrated in Figure 2-10. Dispersion and flocculation represent the extremes in soil fabric classification, and a chemical present in the permeating liquid may influence the permeability of a clay soil by altering the soil fabric toward either of these extremes. Figure 4-1 illustrates how a change in pore diameter can drastically alter permeability.

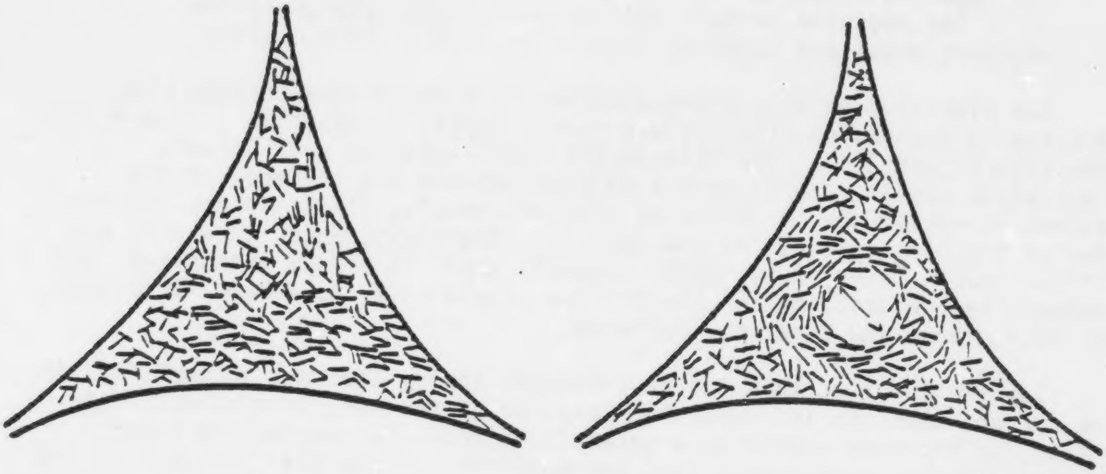
A dispersed deflocculated soil fabric tends to have a large number of very small pore spaces; with flocculation, relatively large-sized interparticle and interaggregate pores are formed. These large diameter pores can cause drastic changes in the permeability of the clay soil since the flow rate is proportional to the square of the diameter of the flow channel.

The pulling together of groups of clay particles into aggregates results when cohesive forces between individual clay particles outweigh the repulsive forces. The forces of attraction result from London-van der Waals forces and do not vary significantly with the chemistry of the pore water. Attractive forces are strongest close to the clay surface and diminish rapidly with increasing distance from the surface.

The forces of repulsion between adjacent clay surfaces, however, are primarily electrostatic and are influenced by the clay surface charge and the chemistry of liquid adjacent to the clay surfaces. Interparticle spacing is a function of the thickness of the diffuse double-layer cationic clouds that form the Gouy layer of the diffuse double layer (Anderson and Jones, 1983). (See also Section 2.3 and discussion below). In theory, the direction of change in permeability associated with varying pore fluid chemistry could be predicted if the variables that affect the thickness of the diffuse double layer are known (Acar and Seals, 1984). However, the heterogeneous mineral composition and wide particle-size distribution common in many soils along with the complicated nature of chemical-soil interactions make this difficult to accomplish in practice.

4.2.1.1 Diffuse Double-Layer Theory--

The theory of the diffuse double layer (also called the electrical double layer) has evolved from the studies of colloid chemistry directed at the description of surface interactions of small particles in a water-electrolyte system. The description that follows is excerpted from Mitchell (1976, pp. 112-113).



Source: Anderson, 1981

Figure 4-1. Change in a pore diameter (400%) corresponding to a permeability increase of 25,600%.

> In a dry clay, adsorbed cations are tightly held by the negatively charged clay surfaces. Cations in excess of those needed to neutralize the electronegativity of the clay particles and their associated anions are present as salt precipitates. When the clay is placed in water the precipitated salts go into solution. Because the adsorbed cations are responsible for a much higher concentration near the surfaces of particles, there is a tendency for them to diffuse away in order to equalize concentrations throughout. Their freedom to do so, however, is restricted by the negative electric field originating in the particle surfaces . . . The negative surface and the distributed charge in the adjacent phase are together termed the DIFFUSE DOUBLE LAYER....

The distribution of cations adjacent to a negatively charged clay particle in suspension (the diffuse double layer) is depicted in Figure 4-2 (see also Figure 2-6). The distribution for a particular soil-water-electrolyte system results from a balance between the tendency of the cations to escape due to diffusion and the opposing electrostatic attraction of the clay surface for the cations. The Gouy-Chapman theory of the diffuse double layer (Gouy, 1910; Chapman, 1913) is widely recognized, and mathematical descriptions of the diffuse double layer have been formulated for both planar and spherical surfaces.

Despite the fact that the Gouy-Chapman theory does not account for all the factors that can influence the behavior of the soil-water-electrolyte system, it has been useful as a generalized model for explaining clay-chemical interactions that affect permeability. Since the thickness of the double layer influences the level of interlayer and interparticle repulsion, system variables that influence the double-layer thickness consequently affect the physical interactions among clay particles.

The nature and thickness of the double layers, and thus the repulsive forces, depend upon characteristics of clay particles and the pore fluid. In general, the tendency for particles in suspension to flocculate decreases with increased thickness of the double layer. An approximate quantitative indication of the relative influences of several factors on the thickness of the double layer is given by Equation (4.3) below (see Mitchell, 1976, p. 118):

$$H = {\frac{DkT}{8\pi n_0 \varepsilon^2 v^2}}^{\frac{1}{2}} , \tag{4.3}$$

where

H = thickness of the double layer

D = dielectric constant of the medium

k = Boltzman constant (1.38×10^{-16} erg/K)

T = temperature in degrees Kelvin

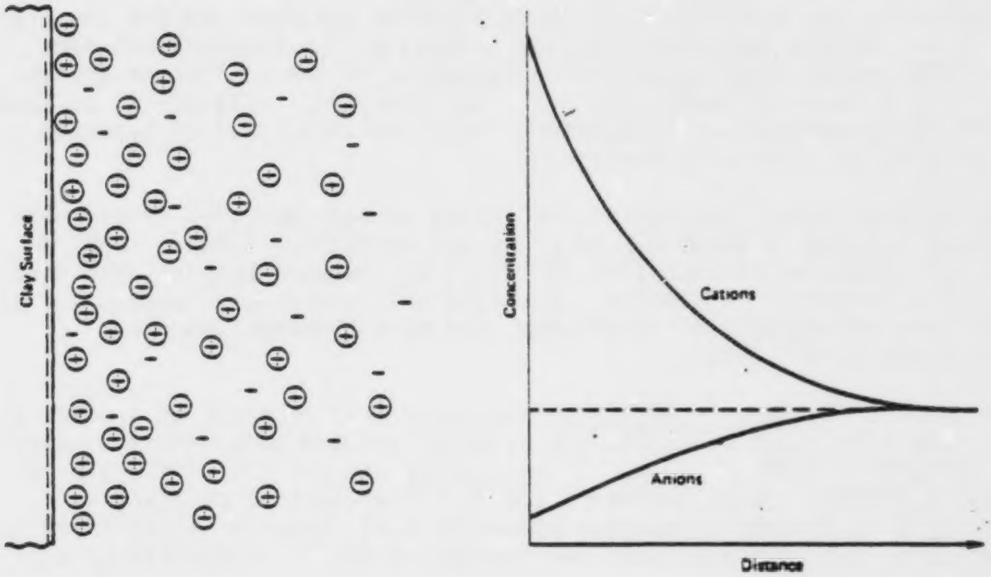


Figure 4-2. Distribution of ions adjacent to a clay surface according to the concept of the diffuse double layer.

n_0 = electrolyte concentration

ε = unit electric charge, 16×10^{-6} coulomb

v = valence of cations in the pore fluid.

The thickness varies inversely with the valence of the cations present and inversely with the square root of the concentration; the thickness increases with the square root of the dielectric constant and the temperature, other factors remaining constant. Based on the Gouy-Chapman model, Lambe (1958) noted that the following variables in the soil-water system affect double-layer thickness and colloidal stability: dielectric constant, electrolyte concentration, temperature, ionic valence, size of hydrated ions present, pH, and anion adsorption.

It has been found that attractive forces exceed repulsive forces when interlayer spacing is about 0.5 nm (Yong and Warkentin, 1975). Thus, a sufficient reduction in repulsive forces (i.e., reduced double-layer thickness) could "transform a massive, structureless, and slowly permeable clay barrier into an aggregated, structured, and more permeable barrier" (Anderson and Jones, 1983).

4.2.1.1.1 Dielectric Constant--The dielectric constant is a measure of the ease with which molecules can be polarized and oriented in an electric field (Mitchell, 1976, p. 113). It represents the ability of a fluid to transmit a charge. Quantitatively, the static dielectric constant is defined by D in Coulomb's equation (Equation 4.4), where F is the force of electrostatic attraction between two charges, Q and Q', separated by a distance d.

$$F = \frac{QQ'}{Dd^2} \ . \qquad (4.4)$$

As the dielectric constant decreases, the fluid film surrounding the clay that contains positive cations must be thinner for the negative surface charge on the clay to be neutralized. For a constant surface charge, the surface potential function will increase as the dielectric constant decreases. Since most organic liquids have dielectric constants substantially lower than water, it is to be expected that the double-layer thickness would be reduced (with an associated tendency toward flocculation) when an organic liquid rather than water surrounds the clay particle. Due to the effects of dielectric constant on the electrical double layer, there is a relationship between the dielectric constant of an adsorbed fluid and interlayer spacing exhibited by clay particles. In general, interlayer spacing decreases with a decrease in the dielectric constant, although this apparent relationship can be complicated by the other factors that affect interlayer spacing.

4.2.1.1.2 Electrolyte Concentration--As the electrolyte concentration in the pore fluid increases, the thickness of the double layer tends to decrease, promoting flocculation. An analysis of the effect of electrolyte concentration on the double layer indicates that an increase in concentration reduces the surface potential for the condition of constant surface charge. Also, the decay of potential with distance is more rapid with increased electrolyte concentration. In essence, the double layer is suppressed by an increase in electrolyte concentration. Interparticle interactions extend to much greater particle spacings for a low electrolyte concentration (e.g., 0.83×10^{-4} M NaCl) than a higher concentration (e.g., 0.83×10^{-2} M NaCl) (Mitchell, 1976, p. 122). The effect of salt concentration on the behavior of clays has been discussed by Anderson (1981):

> As salt concentration in interparticle spaces increases, the cationic cloud is compressed closer to the clay surface, resulting in a decrease in electrostatic repulsion and interlayer spacing. Weiss (1958) noted the direct relationship between salt concentration and interparticle spacing in smectitic clay minerals in a study using distilled water and several concentrations of sodium chloride in water. Both distilled water and 0.01 N NaCl gave infinite interlayer spacing values (the clay was completely dispersed), while 1.0, 3.0, and 5.0 N NaCl gave interlayer spacings of 0.93, 0.61, and 0.58 nm, respectively.

4.2.1.1.3 Temperature--An increase in temperature causes an increase in double-layer thickness with a corresponding tendency toward dispersion. However, the value of the dielectric constant for various fluids is also affected by temperature, generally decreasing with increased temperature. For water, the value of the product DT (Equation 4.3) is reasonably constant, and temperature effects tend to cancel out.

4.2.1.1.4 Ionic Valence--The cation valence affects both the surface potential and the thickness of the double layer. For solutions of the same molarity and a constant surface charge, increasing the cation valence will cause a decrease in the thickness of the double layer and a tendency toward flocculation. It is also shown that an increase in valence will suppress the midplane concentrations between parallel plates, leading to a decrease in interplate repulsion (Mitchell, 1976, p. 122).

4.2.1.1.5 Size of Hydrated Ions--The smaller the size of the hydrated ion, the closer it can approach the surface of the clay particle (Lambe, 1958). Thus, for a given cation valence, the thickness of the double layer will tend to decrease with decreasing hydrated radii of double-layer cations.

4.2.1.1.6 pH--Changes in pH can affect the thickness of the double layer in several ways. The electrolyte concentration as well as the net negative charge on the clay particle are influenced by the solution pH. The formation of stable suspensions or dispersions of clay particles often require high pH conditions. There are two ways in which pH can change the surface charge that results from chemical reaction at the surface of the clay particles. First, a high pH can cause dissociation of hydroxyl groups at the edge of clay particles, increasing the net charge and expanding the double layer. The dissociation reaction is given below:

$$\mathrm{SiOH} \xrightarrow{H_2O} \mathrm{SiO^-} + \mathrm{H^+} \, . \qquad (4.5)$$

The higher the pH, the greater the tendency for the H^+ to dissociate and the greater the effective negative charge. Low pH discourages this dissociation, lowering the surface charge, reducing the thickness of the double layer, and promoting flocculation (Evans et al., 1981; Mitchell, 1976, p. 127).

Second, pH affects alumina exposed at the edges of clay particles. Alumina ionizes positively at low pH and negatively at high pH. Thus, in an acid environment, positive double layers may develop at the edges of clay particles with H^+ serving as the potential determining ion (Mitchell, 1976, p. 126).

Changes in pH do not significantly affect surface charge resulting from isomorphous substitution of the crystal lattice of the clay mineral. Thus, clays with most of their surface charge attributed to isomorphous substitution (e.g., smectites and illites) are less affected by changes in pH than the kaolinite minerals, which have most of their surface charge resulting from surface chemical reactions (see Section 2.3).

4.2.1.1.7 Anion Adsorption--Adsorption of anions (e.g., Cl^-, PO_4^{-3}, and certain surfactants) by the clay particle can increase the net negative charge and increase the double-layer thickness. Dispersion can occur as a result.

4.2.1.2 Displacement of Water--

If adsorbed water within clay particles is displaced by a fluid with a different dielectric constant or electrolyte concentration, the result may be a change in the interlayer spaces of the clay. Such changes can cause the clays to shrink and crack. This can result in the formation of large conducting channels through the soil along with drastic increases in the permeability. Desiccation of clays by certain organic fluids has been reported by Anderson (1981) and others.

4.2.1.3 Cation Exchange--

Cations adsorbed in the diffuse double layer are exchangeable with other cations in solution. In general, the affinity of a cation for a clay increases with cation valence and decreases with increasing ionic radius within an element group. (See Section 2.2.)

Cation exchange will usually result in a change in double layer thickness. The thickness of the double layer decreases with increasing cation valence for montmorillonite; replacement of Na^+ with Ca^{++} results in a reduction of interlayer basal spacing from over 4 nm to about 1.9 nm. For montmorillonite, only two layers of water are incorporated between layers when calcium is the adsorbed cation; with sodium, the number of water layers between layers is practically unlimited. Thus, cation exchange can cause double-layer collapse, desiccation and shrinkage, flocculation, and increased permeability for certain clays.

Theng (1974) has summarized the importance of cation exchange in clay-organic complexes:

> . . . Since uncharged polar organic molecules are adsorbed essentially by replacement of the interlayer water, the behavior of such molecules is likewise strongly influenced by the exchangeable cation. Evidence is accumulating to show that, at least at low water contents, cation-dipole interactions are of paramount importance in their effect on the adsorption of polar organic species by clay materials.

4.2.2 Dissolution by Strong Acids or Bases

Both organic and inorganic acids react with and dissolve aluminum, iron, and silica in the crystal lattice of clay minerals. Strong bases can have a similar effect. This dissolution can result in a release of mineral fragments that may migrate from their original position and leave enlarged pore spaces for conducting the permeant fluid through the clay. Anderson and Jones (1983) have pointed out that "whether the permeability of the clay barrier increases or decreases will depend on the fate of the migrating particles." The increase in conducting pore size may give rise to an increased permeability if the particles migrate through the clay mass. If the particles lodge in pore constrictions, clogging the conducting pore spaces, a decrease in permeability may result (Anderson and Jones, 1983).

Data originally reported by Pask and Davies (1945) show that sulfuric acid dissolves 3, 11, and 89 percent of the aluminum present in kaolinite, illite, and smectite. Other studies also suggest that kaolinite is less soluble than smectite in strong acids (Grim, 1953).

4.2.3 Precipitation of Solids

The precipitation of solids in the soil-water system is controlled by ionic concentration and equilibrium solubility. If the concentration of certain ions exceeds the solubility limits, minerals such as gypsum ($CaSO_4$ $2H_2O$) or jarosite ($KFe_3(SO_4)_2(OH)_6$) may precipitate from solution (Shepard, 1981; Dunn, 1983). Formation of such precipitates can clog pore spaces in the soil matrix.

4.2.4 Effect of Microorganisms

The presence of microorganims can affect the mobility of fluids through the soil. Fuller (1974) classified activity of microorganisms in terms of three classes of chemical reactions: (1) oxidation and reduction, (2) mineralization and immobilization, and (3) reactions with organic constituents. In certain circumstances, the attenuation of contaminants in soils is drastically affected by microorganisms. Processes that can contribute to attenuation of contaminants by microorganisms include the following (Fuller, 1974; Dunn, 1983):

- Degradation of carbonaceous wastes
- Transformation of cyanide to mineral nitrogen and denitrification to nitrogen gas

- Oxidation-reduction reactions with metal ions
- Reduction of sulfate to sulfide
- Production of carbon dioxide from organic molecules
- Production of simple organic acids
- Production of humic and fumaric acids that can react with trace contaminants
- Production of organic species on which trace contaminants can be adsorbed.

Apart from the benefits of attenuation, the physical presence of the microorganisms as well as the gases produced as a result of the reactions listed above can cause blockage in the conducting pore spaces within soils, reducing the area for flow and decreasing permeability. This effect has been noted in permeability tests, particularly when the permeant fluid is conducive to the growth of microorganisms.

4.3 MEASURING CLAY-CHEMICAL COMPATIBILITY THROUGH PERMEABILITY TESTING

Permeability is a highly variable engineering property of soils, and slight changes in the measurement technique or test equipment can cause order of magnitude changes in the values determined (Mitchell et al., 1965). Because one is always dealing with orders of magnitude in permeability testing, it is unrealistic to expect test results to agree within less than several hundred percent (Zimmie, 1981). No widely accepted method exists for measuring clay-chemical compatibility through permeability testing. Thus, a variety of equipment and techniques have been employed in this area of research. Some of the variables in test methods are described in this section, and advantages and disadvantages are highlighted.

4.3.1 Measurement Devices

The permeability of clay soils to various liquids is usually determined in either constant head or falling head tests in fixed-wall, flexible-wall, or consolidation permeameters. Each category of test device can have many variations. Fixed-wall permeameters that have been used in clay-chemical compatibility testing include stainless steel permeameters, plexiglass permeameters, thick-walled pyrex glass permeameters, glass cylinders, PVC (polyvinyl chloride) tubes, and shrink tubing. Shrink tubing may, in fact, lend to fixed-wall tests a major desirable feature of flexible-wall tests--reduction in the possibility of sidewall leakage.

Tests with water and other solutions that do not interact with clays have shown that comparable permeability measurements may be obtained through different types of test devices. Everett (1977) measured the permeability of Lacustrine clays in falling head tests over a 2-month test period in three types of fixed-wall permeameters. Test results showed almost equivalent permeability readings in all test devices. Values measured were 9.0 × 10^{-9} cm/s in a commercial metal permeameter, 6.9 × 10^{-9} cm/s in PVC pipe, and 8.3 × 10^{-9} cm/s in shrink tubing.

Permeability tests performed in both fixed-wall and flexible-wall tests at Duke University (Peirce, 1984; Peirce and Peel, 1985) also indicate that comparable data can be obtained from the different devices. Test fluids used in the Duke investigation did not significantly alter the soil permeabilities as compared to baseline permeabilities obtained with 0.01 N calcium sulfate. More than 100 tests were carried out. The reproducibility of the permeability results suggests that side-wall leakage in fixed-wall devices can be virtually eliminated through careful quality control during sample preparation, compaction, and testing if the permeant fluid does not alter the clay that is tested.

Test results may differ somewhat with different types of permeameters when the test fluid is other than water (or comparable baseline fluid). Daniel (1983) found that methanol in tests with kaolinite in three types of permeameters produced similar curves (permeability versus pore volumes passed through sample) but with compaction mold permeabilities somewhat higher than values measured in flexible-wall or consolidation cells.

4.3.2 Test Setup

Whether permeability tests are carried out as constant head or falling head should not affect the validity of the test results. Effectively, one is doing the same thing in both tests. In the constant head test, readings are converted to a flow rate; in the falling head test, readings are converted to changes in head. Most of the permeability tests described in this section are essentially constant head tests because of the very low permeabilities of compacted clay soils. Test results reported by Monserrate (1982), however, were computed as falling head permeabilities.

Buettner and Haug (1983) noted that leakage is the controlling factor on how low a permeability can be measured. They measured total leakage from their flexible-wall permeameter setup by placing a solid metal block in the cell and monitoring the changes in inflow and outflow. Volume change indicators with an accuracy of ±0.005 cm^3 were used to determine the leakage rate and also enabled the researchers to measure the change in volume of clay soils due to consolidation or swelling. Buettner and Haug found that if leakage is not corrected for, the potential error at low permeabilities (less than 10^{-12} cm/s) can be as high as 2 orders of magnitude.

4.3.3 Compatibility of Materials with Test Fluids

A problem that has become apparent in testing organic fluids in triaxial cells is the incompatibility between the commonly used latex membranes and the fluid to be tested (Acar et al., 1984a; Foreman and Daniel, 1984). Excessive deformations (i.e., wrinkling and expansion) in the membrane may occur with possible effects on the test results. One technique used by

Acar et al. (1984a) to alleviate problems resulting from incompatibility was to wrap the samples with two rounds of 0.03-mm sheet teflon. After placement around the wrapped samples, the membranes were coated with a contaminant-resistant (silicon base) grease to decrease chemical diffusion. Even with these provisions, however, acetone used as the test fluid was found to diffuse through the membrane into the cell water. A procedure and apparatus that can be used to test the membrane-permeant fluid compatibility have been described by Rad and Acar (1984).

Foreman and Daniel (1984) also found a workable technique to be wrapping the soil sample along with the top cap and base pedestal with two or three revolutions of 15-cm (6-inch) -wide Teflon tape before placement of the latex membrane. Long-term flexible-wall tests with heptane have been carried out successfully with this method.

Materials compatibility has also been a problem in fixed-wall tests with corrosive test fluids. Stainless steel permeameters and pressure fittings were damaged in tests with chronic acid (Monserrate, 1982). Several reseachers have used plexiglass devices with teflon fittings to avoid such problems.

4.3.4 Effect of Backpressure

Slight deviations from full saturation have been shown to significantly affect measured permeability values (Mitchell et al., 1965). Recognizing this, some researchers include in the test method a saturation step involving backpressuring. Zimmie et al. (1981) noted that many permeability determinations are made using backpressure to promote complete saturation and then releasing or lowering the backpressure during the permeability test. When this is done, dissolved gases immediately begin to come out of solution, causing the measured permeability values to decrease. Zimmie et al. (1981) concluded, "The maximum, fully saturated permeability value should be determined. It is necessary to utilize backpressure to properly saturate the specimen whether or not the actual permeability test utilizes backpressure."

4.3.5 Effect of Hydraulic Gradient

Mitchell, Hooper, and Campanela (1965), in tests with a silty clay, found evidence that rapid infiltration of water could cause migration of fine particles that tended to plug conducting pore spaces and reduce flow rates. Mitchell and Younger (1967) measured permeability to water of a compacted silty clay as a function of hydraulic gradient. A gradual increase in the gradient from 0 to 17 over 26 days resulted in a corresponding increase in permeability from less than 5×10^{-7} to 5.4×10^{-6} cm/s. The increase was approximately linear in the region of gradients of about 4 to 10, the curve of permeability versus hydraulic gradient tending to flatten at the lower and higher gradients. The nonlinear behavior at the higher gradients may have been the result of movement of fine soil particles. Other researchers--Olsen (1965) and Hamilton (1979)--have reported data that show no significant effect of hydraulic gradient on either discharge velocity or permeability to water.

Under a cooperative agreement with the U.S. Environmental Protection Agency (EPA), Daniel (1982) investigated the effect of hydraulic gradient on permeability. Water (actually 0.01 N calcium sulfate) as well as other permeant fluids were tested at gradients of 10, 50, 100, and 300 in flexible-wall cells, fixed-wall permeameters, and consolidation cells. The permeability of kaolinite to water did not vary with hydraulic gradient in any test device. Changes in hydraulic gradient caused a slight increase in permeability for kaolinite permeated with methanol in flexible-wall cells. In fixed-wall permeameters and consolidation cells, however, the trend was for decreasing permeability with decreasing gradient in tests with methanol. Due to large variations in the test data obtained for fixed-wall permeameters, the results of the tests were somewhat inconclusive.

In other research funded by the EPA, Brown, Thomas, and Green (1984) studied the effects of hydraulic gradient on the permeability of three clay soils to two organic wastes--a xylene waste and an acetone waste. It was concluded that hydraulic gradients of 31, 91, and 181 did not greatly affect the permeabilities in either presaturated or unsaturated samples.

4.3.6 Criteria for Concluding a Test

Clay-chemical compatibility tests should be continued until all changes in permeability resulting from the interaction of the chemical with the clay have been observed. To satisfy this condition, compatibility tests should be concluded only if the slope of the permeability versus time curve does not vary significantly from zero (steady-state permeability has been reached) and at least one pore volume of fluid has passed through the clay. If the first condition is not met, there is no assurance that reactions of the chemical with the clay are complete. If the second condition is not met, there is no assurance that the permeant has contacted all of the clay in the column. In order to satisfy the above conditions, it is necessary to determine the chemistry of both the influent and effluent (to detect breakthrough) and report pore volumes through samples as well as real time. (Conclusion from a workshop on Permeability Testing, Atlanta, GA, January 1984, unpublished.)

A statistical procedure was developed at Duke University (Monserrate, 1982; Peirce, 1984) to determine when a test should be concluded. Readings of column level and time are taken at certain intervals throughout the test. The hydraulic conductivity is computed for each time interval. From this set of data the first 10 points are taken and a linear regression analysis is performed to determine the slope of the hydraulic conductivity vs. time curve. The first point is then dropped and another value is added on the other end. The slope is calculated again, and so on. In the beginning of the test this slope will be fairly large, but as the test progresses it will decrease and approach zero when steady-state is obtained.

The criteria for when steady-state is obtained will be taken when two criteria are met--

- When the slope at the curve does not vary significantly from zero at the 95-percent confidence level, and
- At least one pore-volume of the liquid is passed through the sample (Pierce, 1984).

4.4 SUMMARY OF AVAILABLE RESEARCH DATA

Numerous studies pertaining to the effects of chemicals on clay soil permeabilities have been carried out by researchers at universities and by private firms across the country during the last few decades. Research has been sponsored by the EPA, the Army Corps of Engineers, the Chemical Manufacturers Association, and others. Among the classes of compounds that have been tested are aliphatic hydrocarbons, chlorinated aliphatics, aromatic hydrocarbons, alcohols, glycols, ketones, carboxylic acids, amines, and aromatic nitro compounds. In addition to tests with single-compound test solutions, many studies have been carried out using complex chemical mixtures. In tests involving actual wastes or leachates (obtained by passing water through wastes), the exact composition of the fluid is usually not known although major components and important parameters are usually identified and quantified.

The results of some of the most significant permeability tests involving specific organic solvents are presented in Table 4-1. The results of permeability tests involving wastes are presented in Table 4-2.

Some of the studies summarized here and in more detail in Section 4.5 have shown that certain pure, concentrated organics can drastically alter the permeability of clay soils under the conditions of the laboratory test. These test results have led to widespread concern over the possible seepage of wastes into the environment from clay-lined disposal facilities. It should be emphasized that clay liners in disposal facilities are most often in contact with leachate having much lower organic solvent concentrations than the test solutions associated with the drastic permeability increases seen in some laboratory studies. Laboratory studies performed with less concentrated test solutions (either actual wastes or leachates or dilutions prepared in the laboratory) do not appear to produce such effects.

Because of the wide variations in the soils used in clay liners and the leachates to which they will be exposed during service, testing of the actual liner/leachate system is necessary to confirm compability.

4.5 PERMEABILITY STUDIES TO INVESTIGATE CLAY-CHEMICAL INTERACTIONS

The major findings of various research efforts to investigate effects on permeability of clay-chemical interactions are presented in this section.

4.5.1 Observations by Macey (1942) on Effects of Organics on Fireclay

As a result of permeability experiments with fireclay, Macey (1942) concluded that the rates of flow of benzene, nitrobenzene, and pyridine

TABLE 4-1. RESULTS OF PERMEABILITY TESTS WITH ORGANIC CHEMICALS

ALIPHATIC AND AROMATIC HYDROCARBONS

Heptane

Fixed-wall permeameter tests at high gradient with four clays showed rapid permeability increases and breakthrough. The increase in permeability was more than 3 orders of magnitude in an illite clay. Baseline permeabilities were established in the same sample with 0.01 N calcium sulfate prior to introducing the test solution. (Anderson, 1981)

Benzene

The flow rate of benzene through fireclay was of an enormously higher order of magnitude than the flow rate for water. (Macey, 1942)

In a column test under a head of 701 cm (23 feet), signs of full penetration throughout the clay material were observed after 36 days; in a similar column with water as the permeant fluid the liquid level dropped less than 2.54 cm (1 inch) over a 100-day period. Samples were 91 cm in height and 2.54 cm in diameter. (White, 1976, unpublished data)

Following an initial decrease in permeability (compared to permeability to deionized water established in a similar sample), total breakthrough occurred on the eighth day of testing when Ranger shale was exposed to benzene in a fixed-wall permeameter under low hydraulic gradient. (Green et al., 1979)

Benzene did not penetrate compacted Ca-montmorillonite that was first saturated with 0.01 N calcium sulfate even at hydraulic gradients as high at 150. (Olivieri, 1984)

In flexible-wall tests with a Georgia kaolinite, permeability decreased until the tests were terminated. The final value was approximately 2 orders of magnitude lower than the initial permeability. (Acar et al., 1984a)

Xylene

Following an initial decrease in permeability, total breakthrough occurred on the 25th day of testing when fireclay was tested in a fixed-wall permeameter under low hydraulic gradient. In Ranger shale, a slight decrease in permeability was observed and remained steady until the test was terminated at 40 days. In Kosse kaoline, a slight decrease in permeability was followed by an increase to about the initial level, which persisted until the test was terminated at day 36; changes did not exceed half an order of magnitude. (Green et al., 1979)

In column tests under very low gradient, intrinsic permeabilities were higher than permeability to water by at least 1 order of magnitude in samples of Lake Bottom, Nicholson, Fanno, Chalmers, and Canelo clays. Permeabilities were

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(continued)

TABLE 4.1 (continued)

Xylene (con.)

slightly higher than for water in three other soils. The samples (5.8 cm in diameter and 5 cm in height) were presaturated with xylene. (Schramm, 1981)

Fixed-wall tests at high gradient with four clays showed permeability increases and breakthrough followed by nearly constant permeabilities roughly 2 orders of magnitude higher than baseline permeabilities. Baseline permeabilities were established in the same sample with 0.01 N calcium sulfate prior to introducing the test solution. (Anderson, 1981)

In fixed-wall permeameter tests at high gradient, the permeability of an unsaturated micaceous soil was 4 orders of magnitude higher when exposed to xylene than when tested with 0.01 N calcium sulfate. (Brown et al., 1984)

Xylene/Acetone

In fixed-wall permeameter tests at high gradient, the permeability of an unsaturated micaceous soil to either pure acetone or pure xylene was greater than the permeability determined for mixtures of the two solvents. The permeability of a mixture of 87.5 percent xylene and 12.5 percent acetone was lower by 3 orders of magnitude than the permeability measured with pure xylene (though still higher than the permeability to 0.01 N calcium sulfate). When the acetone component was increased to 75 percent, the permeability was approximately the same as that determined with pure acetone (i.e., about 1.5 orders of magnitude greater than the permeability to 0.01 N calcium sulfate). (Brown et al., 1984)

Kerosene (a mixture of aliphatics and aromatics)

In column tests under very low gradient, intrinsic permeabilities were higher than permeability to water by approximately 1 order of magnitude in samples of Lake Bottom, Nicholson, Fanno, Chalmers, and Canelo clays. Permeabilities were slightly higher than for water in three other soils. The samples (5.8 cm in diameter and 5 cm in height) were presaturated with kerosene. (Schramm, 1981)

In fixed-wall permeameter tests at high gradient, the permeability of an unsaturated micaceous soil increased by 3 to 4 orders of magnitude compared to the permeability determined with 0.01 N calcium sulfate. (Brown et al., 1984)

Naphtha

The permeabilities of two clays (Na-saturated and Ca-saturated montmorillonite) to naphtha were greater by several orders of magnitude than their permeabilities to water. (Buchanan, 1964)

(continued)

TABLE 4-1 (continued)

Soltrol C (a light hydrocarbon liquid)

Intrinsic permeabilities for samples tested with the light hydrocarbon were significantly higher than the permeabilities measured in similar samples exposed to water. (van Schaik, 1970)

Diesel Fuel

Fixed-wall permeameter tests at high gradient with an unsaturated micaceous illite produced highly variable data inconsistent with the pattern of permeability changes seen with other liquid hydrocarbons. Permeability was greater by 1 to 2 orders of magnitude than the permeability measured with 0.01 N calcium sulfate in a similar sample. (Brown et al., 1984)

Paraffin Oil

In fixed-wall permeameter tests at high gradient with an unsaturated micaceous illite, permeability was greater by about 1 order of magnitude than the permeability measured with 0.01 N calcium sulfate in a similar sample. Maximum values were obtained after the passage of one pore volume. (Brown et al., 1984)

Gasoline

In fixed-wall permeameter tests at high gradient with an unsaturated micaceous illite, permeability was greater by 1 to 2 orders of magnitude than the permeability measured with 0.01 N calcium sulfate in a similar sample. (Brown et al., 1984)

Motor Oil

In fixed-wall permeameter tests at high gradient with an unsaturated micaceous illite, permeability increased by about 1 to 2 orders of magnitude as 2.5 pore volumes of fluid were passed through the sample. (Brown et al., 1984)

ETHERS

Dioxane

Kaolinite initially packed and permeated with water was permeated with anhydrous dioxane until complete displacement of the water was achieved. Replacement of water by dioxane was accompanied by about a 20- to 30-percent increase in intrinsic permeability. This permeability was much lower, however, than the values determined for kaolinite beds initially prepared with dioxane. (Michaels and Lin, 1954)

(continued)

TABLE 4-1 (continued)

KETONES

Acetone

In fixed-wall permeameter tests under low hydraulic gradient, three clays showed slight decreases in permeability (compared to permeability to deionized water established in a similar sample). All tests were concluded before 40 days. Less than 0.5 pore volumes were passed through the sample. (Green et al., 1979)

Fixed-wall permeability tests at high gradient with four clays showed initial permeability decreases followed by increases compared to baseline. Baseline permeabilities were established in the same sample with 0.01 N calcium sulfate prior to introducing the test solution. Extensive shrinking and cracking in the soils were observed after permeation. (Anderson, 1981)

In flexible-wall tests with a Georgia kaolinite, an immediate decrease in permeability was followed by an increase, the final value stabilizing at approximately double the initial permeability (Acar et al., 1984)

Acetone (high and low concentration)

An increase over baseline permeability (established with 0.01 N calcium sulfate in similar samples) was seen in an unsaturated micaceous soil for solutions where the acetone concentration was 75 or 100 percent. Samples tested with lower concentrations of acetone did not show appreciable changes in permeability compared to the 0.01 N calcium sulfate. Tests were carried out in fixed-wall permeameters at high gradient. (Brown et al., 1984)

Acetone (low concentration)

Permeability decreased slightly in a Georgia kaolinite clay tested in a flexible-wall permeameter with a solution containing a low concentration of acetone (i.e., below 0.1 percent) prepared in 0.01 N calcium sulfate. (Acar et al., 1984a)

ALCOHOLS, GLYCOLS, PHENOL

Methanol

Permeability decreased slightly (compared to permeability to deionized water established in a similar sample) when Ranger shale was exposed to methanol under low hydraulic gradient in a fixed-wall permeameter. The test was terminated after 30 days. (Green et al., 1979)

Fixed-wall permeameter tests at high gradient with four clays showed steady permeability increases compared to baseline. Baseline permeabilities were established in the same sample with 0.01 N calcium sulfate prior to introducing the test solution. Examination of the methanol-treated samples revealed development of large pores and cracks. (Anderson, 1981)

(continued)

TABLE 4-1 (continued)

Methanol (con.)

Test results with Lufkin clay in flexible-wall cells showed essentially no change in permeability with time when samples were permeated with methanol. The permeability to methanol was virtually the same as with 0.01 N calcium sulfate. (Daniel, 1983)

At high hydraulic gradients, kaolinite was found to have a higher conductivity to methanol than to water regardless of the permeameter types; fixed-wall, flexible-wall, and consolidation permeameters were used. In the flexible-wall and consolidation permeameters, kaolinite is about twice as permeable to methanol as to water. (Foreman and Daniel, 1984)

Isopropyl Alcohol

In column tests under very low gradient, intrinsic permeabilities were higher than permeability to water by almost 1 order of magnitude in samples of Nicholson, Fanno, and Canelo clays. Permeability values were the same or slightly higher than baseline in five other soils. The samples (5.8 cm in diameter and 5 cm in height) were presaturated with the alcohol. (Schramm, 1981)

Glycerol

Permeability decreased slightly (compared to permeability to deionized water established in a similar sample) when Ranger shale was exposed under low hydraulic gradient to glycerol in a fixed-wall permeameter. The test was terminated after 36 days. (Green et al., 1979)

Ethylene Glycol

Fixed-wall permeameter tests at high gradient showed permeability decreases compared to baseline followed by increases in three clays; a smectitic clay showed an initial rapid increase followed by a slower but continuous increase in permeability. Baseline permeabilities were established in the same sample with 0.01 N calcium sulfate prior to introducing the test solution (Anderson, 1981)

In column tests under very low gradient, intrinsic permeabilities were an order of magnitude lower than permeabilities to water in Chalmers clay, Mohave clay, and River Bottom sand. Values were slightly lower in Lake Bottom, Nicholson, Canelo, and Anthony clays and slightly higher in Fanno clay. The samples (5.8 cm in diameter and 5 cm in height) were presaturated with the ethylene glycol. (Schramm, 1981)

(continued)

TABLE 4-1 (continued)

Phenol (949 mg/L)

In column tests with a lacustrine clay (packed to discharge 2 mL/day), no significant effect on permeability was noted when deionized water was replaced by the phenol solution as the permeant fluid. (Sanks and Gloyna, 1977)

Phenol (low concentration)

Permeability decreased slightly in a Georgia kaolinite clay tested in a flexible-wall permeameter with a solution containing a low concentration of phenol (i.e., below 0.1 percent) prepared in 0.01 N calcium sulfate. (Acar et al., 1984a)

Phenol (high concentration)

In flexible-wall tests with a Georgia kaolinite and a high-strength phenol solution, an immediate decrease in permeability was followed by an increase, the final value stabilizing at approximately double the initial permeability. (Acar et al., 1984a)

AMINES

Aniline

Fixed-wall permeability tests at high gradient with four clays showed permeability increases and breakthrough. Baseline permeabilities were established in the same sample with 0.01 N calcium sulfate prior to introducing the test solution. Extensive structural changes in the upper half of the soil columns were observed following permeation with aniline. The aggregated structure was characterized by visible pores and cracks on the surface of the soils. (Anderson, 1981)

Pyridine

The flow rate of pyridine through fireclay was of an enormously higher order of magnitude than the rate of flow for water. (Macey, 1942)

CHLORINATED ALIPHATICS

Carbon Tetrachloride

Permeability decreases slightly (compared to permeability to deionized water established in a similar sample) when Ranger shale was tested under low hydraulic gradient in a fixed-wall permeameter. The test was terminated after 14 days. (Green et al., 1979)

(continued)

TABLE 4-1 (continued)

Trichloroethylene
Permeability decreased slightly (compared to permeability to deionized water established in a similar sample) when fireclay was tested under low hydraulic gradient in a fixed-wall permeameter. The test was terminated after 36 days. (Green et al., 1979)
OTHER
Acetic Acid
Tests at high gradient in fixed-wall permeameters showed continuous permeability decreases to baseline in two clays. Tests with smectitic and illite clays showed permeability increases after initial decreases. Baseline permeabilities were established in the same sample with 0.01 N calcium sulfate prior to introducing the test solution. The permeability decreases were attributed to partial soil dissolution and migration of particles, which temporarily clogged the fluid conducting pores. Progressive soil piping eventually caused the increase in permeability. (Anderson, 1981)
Nitrobenzene
The flow rate of nitrobenzene through fireclay was of an enormously higher order of magnitude than the flow rate for water. (Macey, 1942)
In flexible-wall tests with a Georgia kaolinite, permeability decreased until the tests were terminated. The final value was approximately 2 orders of magnitude lower than the initial permeability (Acar et al., 1984a)

TABLE 4-2. RESULTS OF PERMEABILITY TESTS WITH WASTES

"Xylene Waste" (paint solvent containing xylene with 25 percent paint pigments and trace amounts of water)

In fixed-wall permeameter tests, the permeability of three clay soils, presaturated with 0.01 N calcium sulfate, increased rapidly upon exposure to xylene waste after the cumulative flow exceeded 0.2 to 0.4 pore volume. Permeabilities were 2 to 4 orders of magnitude greater than permeabilities measured with 0.01 N calcium sulfate. Highest permeabilities measured on initially unsaturated samples were greater by 1 to 2 orders of magnitude than for samples that were initially saturated with the calcium sulfate. (Brown et al., 1983)

"Acetone Waste" (a chemical manufacturing waste containing 91.7 percent acetone, 4 percent benzene, and 0.6 percent phenol)

In fixed-wall permeameter tests, the permeability of three clay soils, presaturated with 0.01 N calcium sulfate, initially decreased (minimum permeability at approximately 0.5 pore volume) and then steadily increased. Permeabilities were 2 to 4 orders of magnitude greater than permeabilities measured with 0.01 N calcium sulfate. Highest permeabilities measured on initially unsaturated samples were greater by 1 to 2 orders of magnitude than for samples that were initially saturated with the calcium sulfate. (Brown et al., 1983)

Perchloroethylene Waste

There is evidence that a perchloroethylene waste, which formed a separate, denser than water phase, contributed to the failure of a clay liner at a surface impoundment. (Personal communication, 1984)

"Acid Prowl" (pesticide wash of very low pH; higher viscosity than water)

After several days of exposure to a lacustrine clay packed in a fixed-wall column, the water reacted with the soil to produce chlorine gas. Over a 5-week period, the flow of liquid from the column was irregular due to clogging of pores by the gas. (Everett, 1977)

"Acid Wash" (42 percent sulfuric acid with about 5 percent organics; higher viscosity than water)

Permeability of a lacustrine clay packed in shrink tubing increased by about 1 order of magnitude over a period of 19 days. The permeability was lower, however, than values obtained when the soil sample was exposed to water. (Everett, 1977)

(continued)

TABLE 4-2 (continued)

"Mother Liquor" (an acid wash with pH of 0.37; higher viscosity than water)

Permeability of a lacustrine clay packed in shrink tubing was lower than the value obtained when the soil sample was exposed to water. The waste may have reacted with the soil, liberating gases and increasing pore pressures and clogging flow. (Everett, 1977)

"Hydrazo Benzene" (33 percent methanol, 12.8 percent sodium hydroxide, 15.5 percent sodium formate, and 1.5 percent hydroazobenzene and azobenzene; more viscous than water)

Permeability of a lacustrine clay packed in shrink tubing was slightly lower than the value obtained when the soil sample was exposed to water. Tests were carried out for 34 days under low gradient (less than 100 cm). (Everett, 1977)

"Acid Waste" (100 mM HCl/L)

Due to reaction with carbonates, much higher permeabilities were observed in tests with lacustrine clay compared to permeabilities determined with deionized water. (Sanks and Gloyna, 1977)

"Basic Waste" (100 mM NaOH/L)

Permeabilities in a lacustrine clay decreased compared to the values measured with deionized water. (Sanks and Gloyna, 1977) through clay are "of an enormously higher order than for water." Analytical reagent-grade organics were used. All the organic liquids passed through the clay at flow rates 100,000 to 1,000,000 times greater than the flow rate of water through the same clay. The high rates of flow produced experimental difficulties because of the high resistance to flow of the testing apparatus, and the experiments were finally abandoned. Differences between the viscosities of the organic fluids and of water are not sufficient to account for the very large permeability changes that were observed.

4.5.2 Tests With Kaolinite and Organic Solvents by Michaels and Lin (1954)

Michaels and Lin (1954) measured the permeability of high-purity kaolinite to nitrogen gas, cyclohexane, acetone, dioxane, methanol, and distilled water. Tests were also conducted to determine the effect of replacement of one permeant liquid by another. The investigation was one part of a broad program of fundamental research sponsored by the U.S. Army Corps of Engineers and by industrial contributions and was carried out at the Massachusetts Institute of Technology (MIT) Soil Stabilization Laboratory.

4.5.2.1 Test Method--

Permeability tests were conducted in a 5.1-cm (2-inch) -diameter fixed-wall permeameter cell. Bed thickness was 1.1 to 1.8 cm. The base-exchange capacity of the kaolinite was determined to be 5.0 ± 0.5 meq/100 g. Samples were dried for 24 hours at 160 °C prior to saturation by the organic liquids for the permeability test.

Prior to permeability testing, each sample was submerged in the fluid to be tested (either organic liquid or water). The wetted samples containing known amounts of clay were poured into the permeability cell, entrapped air was removed, and the sample was consolidated by means of a confining piston. Samples were consolidated to several void ratios (volume of voids/volume of solid) for each test fluid. Void space was determined through column bulk density and the true density of kaolinite.

For the permeability tests, a hydrostatic head not exceeding 10 percent of the compaction pressure was maintained. Permeabilities were determined for each void ratio. Permeability coefficients determined for separate clay samples prepared with a given fluid and compacted to essentially the same void ratio were found to be reproducible to within 10 percent.

The effect of change of fluid medium on the permeability of a confined clay was determined through a series of desolvation experiments. Kaolinite initially packed and permeated with water was subsequently permeated with dioxane until the water was completely displaced (as evidenced by constant permeability to dioxane). The dioxane was then displaced with acetone under similar conditions. Finally, warm, dry nitrogen was passed through the sample until the bed was solvent-free. Beds initially tested with the organic liquids (methanol, dioxane, acetone, and cyclohexane) were similarly treated with dry nitrogen until all solvent was removed.

The permeability to nitrogen of the desolvated samples was then determined and compared to the permeability with the original permeant fluid.

4.5.2.2 Test Results--

The intrinsic permeabilities (cm^2) of samples packed at different void ratios, measured for organic solvents and for nitrogen, are shown in Figure 4-3. Predictably, the permeabilities increased with increasing void ratio. The data also indicate that permeability at any given void ratio decreases with increasing polarity of the permeant.

As a result of the desolvation experiments, the authors concluded that:

> Whatever specific effects these permeant liquids exert on the kaolinite persist when the clay is thoroughly dried in the confined state. The reduced permeabilities of kaolinite to these liquids relative to the values observed for this clay to gas when packed in the dry state apparently cannot be ascribed to adsorbed liquid films, abnormally high liquid viscosities, or electro-osmotic effects. . . . It seems most probable, therefore, that the lower permeabilities of kaolinite to these liquids than to nitrogen are caused by improved dispersion, and possibly more orderly packing of the solids in more polar media.

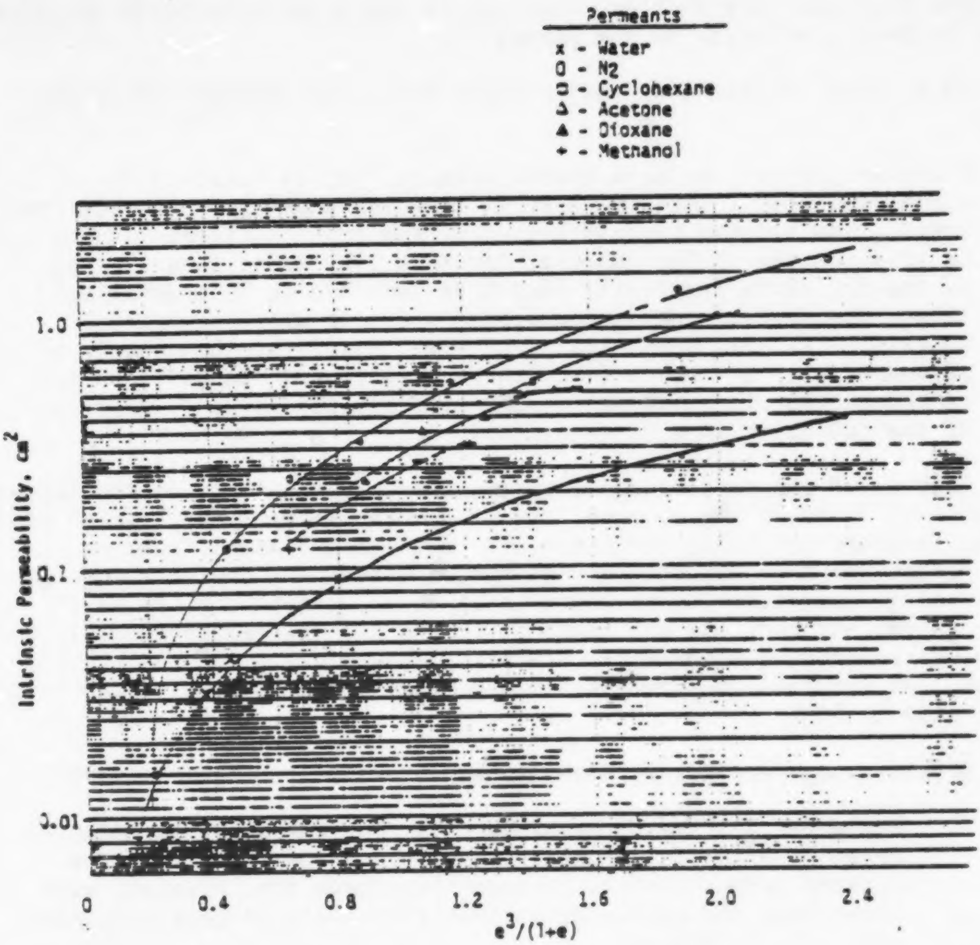
Results of the desolvation experiments show the effect of replacement of permeant fluid on the permeability of the kaolinite. For samples sedimented from and initially permeated with methanol, dioxane, acetone, or cyclohexane and then exhaustively permeated with warm, dry, nitrogen gas, the final gas permeability is found to be equal, within the limits of experimental accuracy, to the permeability to the initial permeant liquid at the corresponding void ratio. This is particularly true at low void ratios. An increase in permeability upon drying is observed if the initial void ratio is high.

Specific observations by Michaels and Lin include the following:

- Replacement of water by dioxane is, in all cases, accompanied by about a 20- to 30-percent increase in permeability; the dioxane permeabilities through these beds are, however, much lower than the values determined for kaolinite beds prepared initially in dioxane.

- Replacement of the dioxane with acetone leads to a small additional permeability increase approximately when water is the initial permeant fluid.

- Displacement and evaporation of the acetone with dry nitrogen is accompanied by a great increase in permeability; the magnitude of this increase is greatest for kaolinite samples confined initially at high void ratios.



Source: Michaels and Lin, 1954

Figure 4-3. Intrinsic permeabilities as a function of void space (e) measured for different permeants.

4.5.2.3 Discussion--

Intrinsic permeabilities reported by Michaels and Lin cannot be directly compared with permeability data from other tests because the measurement procedure is unique. No information is provided about the amount of fluid passed through the sample in determining each of the reported permeabilities.

While samples used in permeability tests are usually air-dried, the clay samples used by Michaels and Lin were dried for 24 hours at 160 °C, a condition that could cause irreversible changes in clays and organic matter. It is possible that the adsorptive properties of the kaolinite were permanently altered by this drying procedure. The kaolinite mineral, halloysite, will not rehydrate if the interlayer water is removed.

4.5.3 Study by Buchanan (1964) of the Effect of Naphtha on Montmorillonite

Buchanan (1964) at Texas A&M University studied permeabilities of sodium- and calcium-saturated montmorillonite when these clays were permeated by water and naphtha. Permeabilities of the two clays to naphtha were greater by several orders of magnitude than their permeabilities to water, even with reductions in void ratios for the samples treated with naphtha. The data also suggest that the permeability of the sodium-saturated clay is more dramatically affected by the naphtha. Permeability differences were thought to be at least partially due to the inability of naphtha to form an immobilized liquid film on the clay mineral surfaces. Buchanan's data are summarized in Table 4-3.

4.5.4 Study by Reeve and Tamaddoni (1965) of the Effect of Electrolyte Concentration on Permeability of a Sodic Soil

Reeve and Tamaddoni (1965) of the U.S. Department of Agriculture studied the effects of high-salt, high-sodium solutions on the permeability of a highly sodic soil. California Waukena clay loam containing about 15 percent of expanding lattice-type clay was tested in the laboratory and in the field.

4.5.4.1 Test Method--

Solutions of varying concentrations but with constant sodium adsorption ratio (SAR) were used in the tests. Test solutions were prepared from sodium chloride (NaCl) and calcium chloride ($CaCl_2$) at SAR levels of 0 (100 percent $CaCl_2$), 80 (a partially reclaiming solution), 180 (the equilibrium solution of the natural soil), and ∞ (100 percent NaCl). For each SAR level, seven ionic concentrations (63, 125, 250, 500, 1,000, 2,000, and 4,000 meq/L) were tested in a selected sequence. Laboratory tests were carried out in fixed-wall 5.4-cm (2.125-inch) ID cylinders.

4.5.4.2 Test Results--

The test results indicate that permeability is a function of both the absolute concentration and the SAR of the initial solution. Permeability increased with increasing SAR value and ionic concentration. The permeability-concentration relationship was found to depend markedly on the initial solution concentration (i.e., if the initial solution concentration was

TABLE 4-3. VOID RATIO AND COEFFICIENT OF PERMEABILITY RELATIONSHIPS FOR CALCIUM- AND SODIUM-MONTMORILLONITE PERMEATED BY WATER AND NAPHTHA[a]

Clay	Water		Naphtha	
	Void ratio	Permeability (cm/s)	Void ratio	Permeability (cm/s)
Calcium-saturated smectite	1.72	1.6×10^{-9}	1.52	6.4×10^{-5}
Sodium-saturated smectite	3.75	5.2×10^{-11}	1.31	3.8×10^{-5}

[a] Data from Buchanan (1964).

TABLE 4-4. SUMMARY OF SOIL PERMEABILITY WITH SOLTROL C AND WATER[a]

Soil (bulk density in g/cm³)	Intrinsic permeability (um²)		Permeability, K (cm/s)	
	Water	Oil	Water	Oil
Cavendish loamy sand (bulk density: 1.44 g/cm²; 6 percent clay)	4.94	9.38	4.8×10^{-3}	3.0×10^{-3}
Chin sandy clay loam (bulk density: 1.25 g/cm³; 18 percent clay)	0.73	5.45	7.1×10^{-4}	5.1×10^{-3}
Chin sandy clay loam (bulk density: 1.38 g/cm³; 18 percent clay)	0.22	2.53	2.1×10^{-4}	1.4×10^{-3}
Lethbridge clay loam (bulk density: 1.22 g/cm³; 37 percent clay)	0.51	6.10	5.0×10^{-4}	3.3×10^{-3}

[a] Data from van Schaik (1970).

low, the range of permeability values with varying solution concentrations was lower than if the initial solution concentration was high, and vice versa.) Apparently the degree of flocculation or dispersion that occurred with the application of the initial solution influenced the magnitude of the permeability.

In some of the tests a large change in solution concentration (e.g., from 125 to 1,000 meq/L) was accompanied by a large increase in outflow due to sidewall leakage as a consequence of the contraction of the previously swollen mass.

In the test series in which initially low concentrations (63 meq/L) were increased stepwise to 4,000 meq/L and then reduced to the initial concentration, reproducibility of permeability values as a function of concentration was good. These results suggested that a reversible swelling process was occurring. In the test series in which initial concentrations of 1,000 meq/L were increased to 4,000 meq/L and then reduced to 63 meq/L, results showed a gradual decrease in permeability with a given solution concentration. This was attributed to slaking or particle rearrangement that led to permeability decreases that progressed with time.

The permeability of the Waukena soil to a saturated calcium sulfate ($CaSO_4$) solution was an order of magnitude lower than the lowest values measured with the test solutions.

Test results showed that intake rates measured in the field at high electrolyte concentrations were approximately three times as great as corresponding permeabilities measured in the laboratory. However, the fractional change was essentially the same at equal electrolyte concentrations.

4.5.5 Tests by van Schaik and Laliberte (1968) of Permeability of Soils to a Liquid Hydrocarbon

Permeability data are reported by van Schaik (1970) for three soils tested with tap water and with a light hydrocarbon liquid (trade name Soltrol C). Permeability measurements were made on samples that had been vacuum-saturated. The intrinsic permeabilities with the oil were first reported by van Schaik and Laliberte (1968).

4.5.5.1 Test Results--

The saturated intrinsic permeability of the three samples varied between 0.22 and 4.94 μm^2 for water. For oil, the values ranged between 2.53 and 9.38 μm^2. The data are summarized in Table 4-4.

4.5.5.2 Discussion--

The test apparatus used in the study was designed to measure properties of unsaturated porous media. The number of pore volumes displaced by the test fluid during the permeability determination was not reported. Details of the test procedure used also were not given in the references cited. In spite of these limitations, the intrinsic permeabilities are useful for comparing the relative permeabilities of the various soils to water and oil. The increased permeabilities of the soils when exposed to the oil are consistent with the later findings by Anderson (1981).

4.5.6 Study by Everett (1977) of Permeability of Lacustrine Clay to Four Liquid Wastes

Everett (1977) investigated the permeability of lacustrine clays from Bay County, Michigan, to four liquid wastes. The purpose of the study was to establish the feasibility of a hazardous waste landfill in Bay County. Three types of permeability test devices were used. The results of the comparative tests with water show no significant differences in the permeabilities measured in the different devices.

The predominant mineral in the soil was quartz, estimated to be 40 to 50 percent. Clay minerals present were illite and chlorite, each at about 10 to 15 percent. Other important constituents were dolomite (20 to 25 percent) and calcite (10 to 15 percent). The clay fraction was reported to average 18.1 percent. The mean cation exchange capacity of the soil (determined with ammonium acetate solution) was low.

Wastes used in the compatibility tests were "Acid Prowl," "Acid Wash," "Mother Liquor," and "Hydrazo Benzene." "Acid Prowl" is a pesticide wash of very low pH. The "Acid Wash" was about 42 percent sulfuric acid in water and also contained about 5 percent organics including dichlorobenzidine, orthochloroaniline, and tars. The waste labeled "Hydrazo Benzene" actually contained about 1.5 percent hydroazobenzene and azobenzene. Other components were methanol (33 percent), sodium hydroxide (12.8 percent), sodium formate (15.5 percent), and water (36.7 percent). The waste has a pH of 12.10. The "Mother Liquor" was an acid wash (pH of 0.37) of unknown use. All waste samples were more viscous than water.

4.5.6.1 Test Method--

Permeability to water was measured in three different test devices--commercial Soil Test® units (fixed-wall, 10.2-cm [4-inch] diameter, 10.2-cm [4-inch] sample height), PVC pipe (10.3-cm [4-inch] diameter, 15.2-cm [6-inch] sample height), and shrink tubing (10.2-cm [4-inch] diameter, 15.2-cm [6-inch] length). Samples tested in the Soil Test units and the shrink tubing were prepared in a standard Proctor mold according to ASTM specifications for modified Proctor compaction (D-1557-70). The objective of preparing the samples was to obtain uniform packing. The PVC pipe was packed in a different manner, not specified.

Samples were saturated by backwashing with water for more than 2 weeks before permeability data were recorded. Readings were taken over a 2-month period to determine the permeability of the samples to water. After testing with water, three shrink tube columns and the PVC pipe columns were tested with wastes. Samples were allowed to saturate by backwashing with the waste until the effluent showed the presence (by color or pH) of the waste.

The tests were all conducted in falling head setups with a head of less than 100 cm (39.37 inches). The metal test units were not used for testing the wastes because of the corrosive nature of the wastes.

4.5.6.2 Test Results--

"Acid Prowl" was tested in the PVC column. After several days of contact with the soil, chlorine gas became noticeable at the outlet pipe. After 2 weeks, the gas was no longer apparent, and liquid flow from the permeameter ceased. The column was allowed to stand with the waste fluid head for 3 more weeks. During this time the fluid reacted irregularly. This behavior was attributed to clogging of pores by chlorine gas generated with the sample. Valid permeability readings could not be obtained.

"Acid Wash," "Mother Liquor," and "Hydrazobenzene" were tested in shrink tubes. The permeabilities measured with the wastes were lower than the corresponding permeabilities to water. This may be due in part to the higher viscosity of the wastes compared to water. Values obtained after 19 days of testing with "Acid Wash" showed a slight increase in permeability (about 1 order of magnitude) compared to the initial value. For "Mother Liquor," test data with the waste showed permeability to be lower by more than 2 orders of magnitude than values measured with water. These acid wastes may have reacted with the soil-liberating gases, which would increase pore pressures and consequently decrease the permeability. In tests with hydrazobenzene, permeability values were slightly lower than values obtained with water. Permeability data are compared in Table 4-5.

4.5.6.3 Discussion--

Although the compatibility tests were carried out for approximately 1 month, the volumes of liquid forced through the columns under the low gradients had to be very small. The number of pore volumes displaced was not specified. Sufficient data were not presented to determine quantitatively the effect attributable to the viscosity of the wastes.

4.5.7 Tests by Sanks and Gloyna (1977) of Permeability of Lacustrine Clay to Liquid Waste

Sanks and Gloya (1977) tested five simulated liquid wastes with three clays in column tests to determine effects on permeability. The aqueous waste solutions tested were chosen to be representative of materials that might leak from containers of solid wastes. The various substances contained acid, base, and heavy metals. Clays tested contained large percentages of montmorillonite. The columns were packed at densities low enough to discharge 2 mL/day of distilled water at 152-cm (5-ft) head with an allowable error of 10 percent. After the tests with water, the columns were drained and refilled with the waste fluid to be tested.

Permeabilities with deionized water ranged from 4×10^{-8} to 1.2×10^{-7} cm/s. Much higher permeabilities were observed when the acid waste (100 mM HCl/L) was passed through the columns due to reaction with carbonates. Permeabilities decreased in tests with the basic waste (100 mM NaOH/L). Phenol at 10 mM/L (940 mg/L) appeared to have little effect on permeability.

TABLE 4-5. PERMEABILITIES MEASURED WITH LACUSTRINE CLAY EXPOSED TO WATER AND WASTE LIQUIDS[a]

Day of test data	Permeability (cm/s)		
		With water	
1	3.4×10^{-10}	6.7×10^{-7}	4.0×10^{-7}
3	1.3×10^{-9}	6.5×10^{-7}	6.0×10^{-7}
5	1.5×10^{-9}	5.7×10^{-7}	5.1×10^{-7}
	With "acid wash"	With "mother liquor"	With "hydrazobenzene"
15	7.1×10^{-11}	9.0×10^{-10}	9.0×10^{-8}
19	No flow	1.0×10^{-9}	7.8×10^{-8}
29	2.9×10^{-10}	8.3×10^{-10}	No reading
32	5.8×10^{-10}	2.1×10^{-9}	1.3×10^{-7}
34	6.9×10^{-10}	3.1×10^{-10}	1.3×10^{-7}

[a] Data from Everett (1977).

Heavy metals tested, $HgCl_2$ at 30 mM/L (8,100 mg/L) and $ZnSO_4 \times 7H_2O$ at 30 mM/L (8,610 mg/L), showed no effect except for one anomaly.

4.5.8 Investigation of the Effect of Organic Solvents on Clays by Green and Jones (1979)

Under a grant from EPA's Kerr Environmental Laboratory, Ada, Oklahoma, to the University of Texas at Dallas, Green, Lee, and Jones (1979) studied the impacts of organic solvents on the shrink/swell behavior and permeability of four clay soil materials--Ranger shale, fire clay, Kosse kaoline, and Parker soil. The test results were compared to results with water. The studies included extensive soil characterization. Soil properties are summarized in Table 4-6. Organic solvents tested were benzene, xylene, carbon tetrachloride, trichlorethylene, methanol, and glycol (Green et al., 1981, 1983).

4.5.8.1 Test Methods--

Swell properties of the clays in contact with water, the organic solvents, and various solvent mixtures were investigated. The swell properties were measured with a 6.4-cm-diameter consolidometer. Care was taken to prevent evaporation. Clay core samples used in the tests were compacted at optimum moisture content. In the consolidometer, samples were flooded with the test fluid and measurements of the swell or shrink behavior were made as a function of time.

Permeability data were collected on 15 clay/solvent systems. Test durations ranged from 8 to 40 days. Fixed-wall permeameters used in the study were thick-walled Pyrex glass, and all joints were teflon-lined. The test procedure was adapted from an ASTM method. Samples were compacted at optimum moisture conforming to standard compaction procedures before transfer to the permeameters. The test fluid was then introduced, the liquid level being adjusted in an 8-mm graduated standpipe. Equilibrium permeabilities were estimated from curves of K(cm/s) versus time. The number of pore volumes of permeant fluid passed through the samples was not reported.

4.5.8.2 Test Results--

Each of the clay soils swelled to a greater extent when exposed to deionized water than with any of the pure solvents tested. The shrink/swell behavior of the various clays in water and in organic solvents is described in Table 4-7. The final percent swells for the samples are listed in Table 4-8.

Based on the swelling data for both pure solvents and for mixtures, the authors concluded that "in mixtures of solvents, a clay will tend to swell as though it were immersed in the component of higher dielectric constant only" (Green et al., 1979).

When Ranger shale was presented with benzene, breakthrough occurred on the eighth day of testing. Breakthrough occurred at about day 25 in the fire clay sample presented with xylene.

TABLE 4-6. PROPERTIES OF SOILS TESTED[a]

	Clays			
Properties	Ranger shale	Fire clay	Kosse kaoline	Parker soil
Particle size distribution (wt. %)				
Clay	40	44	53	10.5
Silt	59	55	47	70.5
Sand	1	1.5	0	19
Total carbon (%)	0.60	0.03	0.12	3.18
Carbonate (%)	0.32	0	0	0.42
Cation exchange capacity	54.4	11.2	13.4	14.1
Atterberg limits				
Plastic limit (moisture %)	36	31	38	22
Liquid limit (moisture %)	46	32	50	26
Optimum moisture content (%)	17.5	16	31	18
Corresponding dry density (g/cm^3)	1.73	1.81	1.36	1.86
Mineralogy (% of clay fraction)				
Kaolinite	24	78	85	14
Quartz	28	16	10	Negligible
Illite/mica	24	10	5	37
Chlorite-montmorillonite	10	Negligible	Negligible	49

[a] Data from Green et al., 1979.

TABLE 4-7. CLASSIFICATION OF CLAY-ORGANIC SOLVENT SYSTEMS ACCORDING TO SWELL PROPERTIES[a]

Swelling only	Swelling then shrinking	Shrinking then swelling	Shrinking only
RS/H_2O	RS/acetone	RS/benzene	RS/xylene (NS)
RS/glycerol	KK/xylene	KK/TCE	KK/CCl_4 (NS)
RS/methanol	FC/acetone	FC/benzene	FC/CCl_4 (NS)
RS/CCl_4	FC/TCE		
RS/TCE	FC/xylene (NS)		
KK/water			
KK/acetone			
FC/water			

RS = Ranger shale
FC = Fire clay
NS = Net shrinkage (net swell observed unless indicated otherwise)
KK = Kosse kaoline
TCE = Trichloroethylene

[a] Reproduced from Green, Lee, and Jones, 1979.

TABLE 4-8. PERCENT SWELL FOR CLAY SOILS IN CONTACT WTH ORGANIC LIQUIDS AND WATER[a]

Clay-soil	Solvent	Percent swell[b]
Ranger shale	Benzene	0.05
	Benzene/acetone (3:1)[c]	5.75
	Xylene	-0.11
	Carbon tetrachloride	1.1
	Trichloroethylene	1.0
	Acetone	4.0
	Acetone/benzene (3:1)[c]	4.6
	Acetone/water (1:1)[d]	11.0
	Methanol	11.4
	Glycerol	5.3
	Water	11.7
Kosse kaoline	Xylene	0.16
	Acetone	8.7
	Water	11.7
Fire clay	Xylene	-0.25
	Carbon tetrachloride	-0.6
	Acetone	3.6
	Water	8.2

[a] Adapted from Green et al., 1979.

[b] Negative value indicates net shrinkage.

[c] Mole percent.

[d] Volume percent.

For organic solvents other than benzene, a decrease in permeability of Ranger shale was observed for 5 to 10 days. This decrease was followed by stable readings at a minimum value. The permeability coefficients reported are shown in Figure 4-4. Although there is some scatter in the measurements characteristic of normal experimental error, the decreasing permeability trends over the duration of the test period are evident. The largest decrease in permeability compared to water is seen with glycerol. With carbon tetrachloride, the decrease in permeability was followed by an increase that continued to the end of the test period.

4.5.8.3 Discussion--

The authors based their final conclusion, an empirical relationship for estimating the coefficient of permeability from dielectric constant of the permeant and packed bulk clay density, on what they judged to be final permeabilities. Their data do not indicate that stable permeabilities were reached, however, in all of the solvent clay systems tested. Certainly the permeabilities measured just prior to breakthrough could not be classified as stable, yet these were used in obtaining the empirical equation to predict permeability.

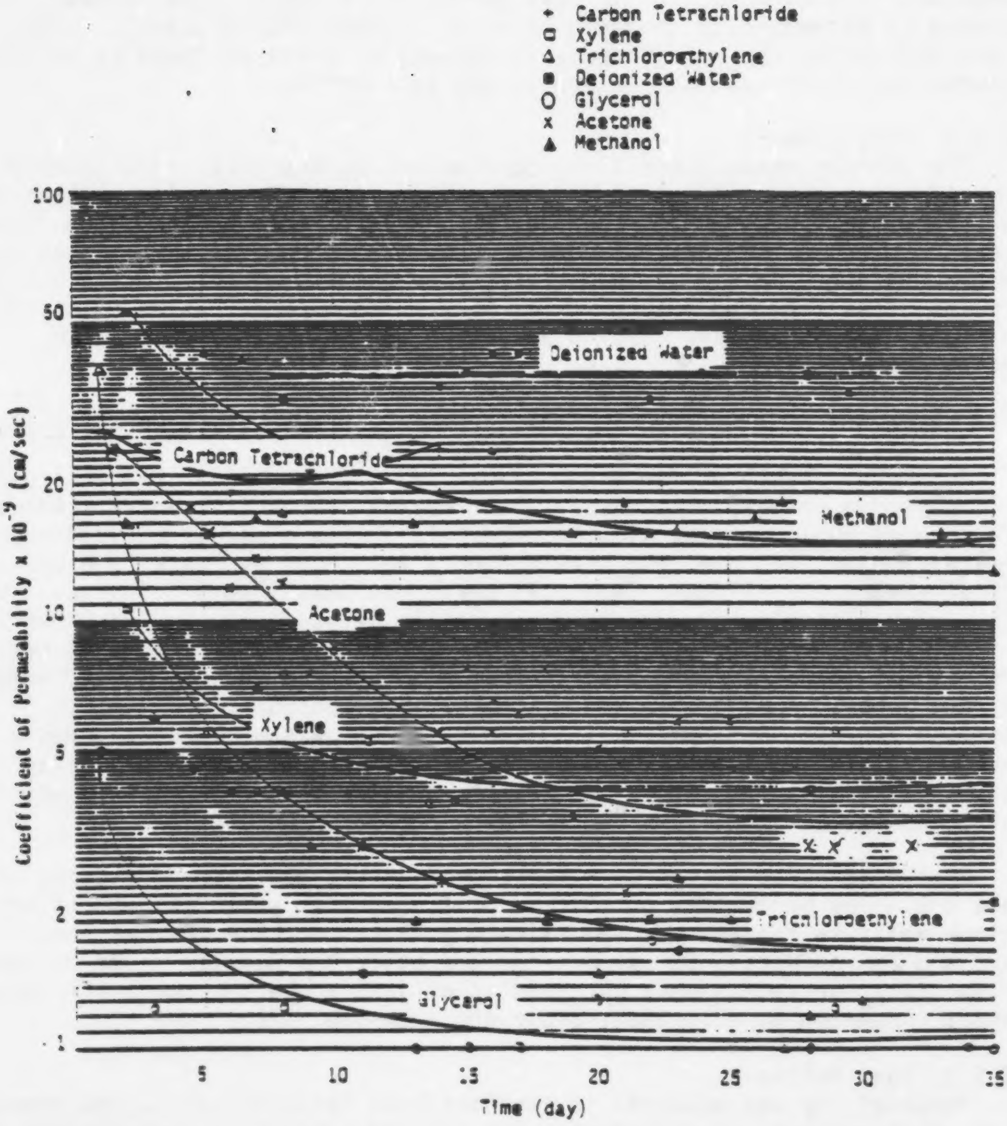
4.5.9 Anderson's Study (1981) of the Effects of Organics on Permeability

Anderson (1981) at Texas A&M University studied seven organic fluids in comparative permeability tests. Four native clay soils were tested--two with predominantly montmorillonite (smectitic) clay minerals but different chemical properties, one with predominantly kaolinite minerals, and one with predominantly illite. Each soil contained a minimum of 35 percent by weight clay minerals and exhibited a baseline permeability of less than 1 × 10^{-7} cm/s when compacted at optimum water content. Organic solvents tested were reagent grade ethylene glycol, acetone, heptane, xylene (mixed isomers), aniline, glacial acetic acid and methanol. The control fluid used to establish the baseline permeability for each soil was a standard aqueous solution of calcium sulfate (0.01 N $CaSO_4$). (Although it was not noted in the initial publication of this research, the methanol used in the experiment contained 20 percent water [Anderson 1982].)

Characteristics of the four soils used in the tests are presented in Table 4-9. The percentage of soil voids filled with water at compaction at optimum moisture content was approximately 75 percent for the two smectitic clays and approximately 90 percent for the other two samples. The minimum permeability for each sample occurred at or just above the optimum moisture content (Anderson et al., 1981; Brown and Anderson, 1982).

4.5.9.1 Test Method--

Permeability was measured in constant head tests on soil cores compacted at or above optimum water content. Pressurized compaction permeameters with an air-induced elevated hydraulic gradient were used. For two montmorillonite clays, a gradient of 361.6 (equivalent to a hydraulic head of 42.2 m of water) was used. For the illite and kaolinite clay soils, a hydraulic gradient of 61.6 (equivalent to a hydraulic head of 7 m of water) was imposed. No signs of particle migration or turbulent flow resulted from the elevated gradients.



Source: Green, Lee, and Jones, 1979

Figure 4-4. Coefficient of permeability of Ranger shale to various chemicals.

TABLE 4-9. GRAIN SIZE DISTRIBUTION, MINERALOGY, AND PROPERTIES OF THE FOUR CLAY SOILS[a]

Clay soil description	Non-calcareous smectite	Calcareous smectite	Mixed cation kaolinite	Mixed cation illite
% Sand (>50 nm)	35-37	7-8	39-41	14-15
% Silt (50-2.0 nm)	26-28	42-44	17-18	38-39
% Clay (<2.0 nm)	36-38	48-50	42	47
Coarse clay (2.0-0.2 nm)				
% of total	16	25	33	61
Mineralogy[b]	QZ-1 KK-2 MI-2	MT-1 KK-2 QZ-3	KK-1 QZ-2	I-1 QZ-2
Fine clay (<0.2 nm)				
% of total	84	75	67	39
Mineralogy[b]	MT-1	MT-1 KK-3	KH-1 MI-3	I-1 MT-2
Cation exchange capacity (meq/100 g)	24.2	36.8	8.6	18.3
Total alkalinity (meq/100 g)	3.3	129.2	0.8	4.2
Fe_2O_3 (%)	0.42	0.2	13.2	-
Organic matter (%)	0.9	3.0	0.6	-
$CaCO_3$ Equiv. (%)	-	33	Trace	-
Shrink-swell potential	Very high	Very high	Moderate	Moderate
Liquid limit	51-67	58-98	41-60	~46
Plasticity index	30-45	34-72	18-30	~27
Optimum water content[c]	20.0	21.5	20.0	19.0
Maximum density (kN m^{-3})	15.0	14.4	16.3	16.6

[a] Adapted from Anderson, 1981.

[b] Key to mineralogy:
MT = Smectite
KK = Kaolinite
I = Illite
1 = >40%
2 = 10-40%
3 = <10%
QZ = Quartz
MI = Mica or illite
KH = Halloysite

[c] Percent by dry weight.

Special precautions were taken to minimize several sources of error that occur frequently in permeability testing. At least one pore volume of the $CaSO_4$ solution was passed through the soil cores to minimize trapped air. Before the permeameter was pressurized, the top of the soil sample was exposed to 10 cm of standard $CaSO_4$ solution for 48 hours. This procedure helps to prevent channel formation and bulk flow and promotes sealing of the permeameter sidewalls. Provision was made to view the effluent passing through the soil sample so that trapped air or evidence of piping could be monitored. Care was taken to minimize evaporative losses from the collected effluent. Teflon fittings were used wherever possible to avoid deterioration from exposure to the organic solvents.

After seating the soils at low pressure, the selected gradient was applied to the permeameter fluid chamber until stable permeability values were obtained with the standard $CaSO_4$ solution. At this point, pressure was released and permeameters were disassembled. Soil that had expanded out of the mold was removed and additional standard leachate was passed through the remaining sample in the permeameter. Percent swelling was determined for each soil. At this point, the soil sample was assumed to be completely saturated.

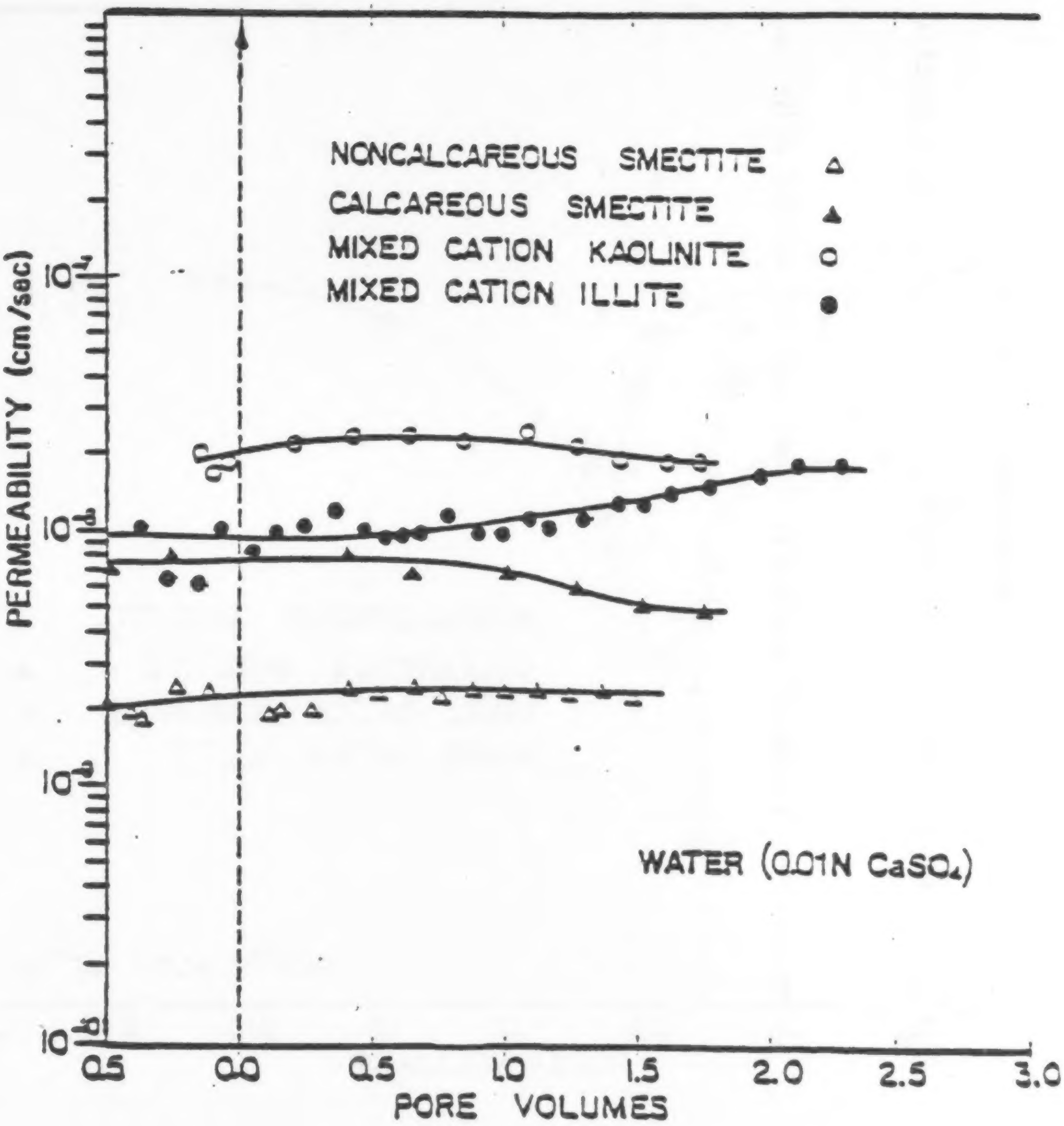
After the saturation step to ensure stable baseline permeability, the remaining standard $CaSO_4$ solution was removed and replaced with one of the organic solvents to be tested. The selected gradient was imposed and between 0.5 and 2.0 pore volumes of fluid were passed through the sample in the permeameter. The permeameters were then depressurized, disassembled, and the cores removed and examined for evidence of structural changes. Following the permeability tests of the noncalcareous smectite clay soil with aniline, methanol, and ethylene glycol (all highly water-soluble), water was reintroduced to the test columns and permeabilities were again determined.

During the permeability tests, effluent was carried to an automatic fraction collector, which separated the samples obtained from 10 permeameters during specific time increments. The percentage of organic fluid in the effluent samples was determined to develop breakthrough curves. A permeability value, leachate volume, and time increment were obtained on each volume of fluid passed through a sample.

4.5.9.2 Test Results--

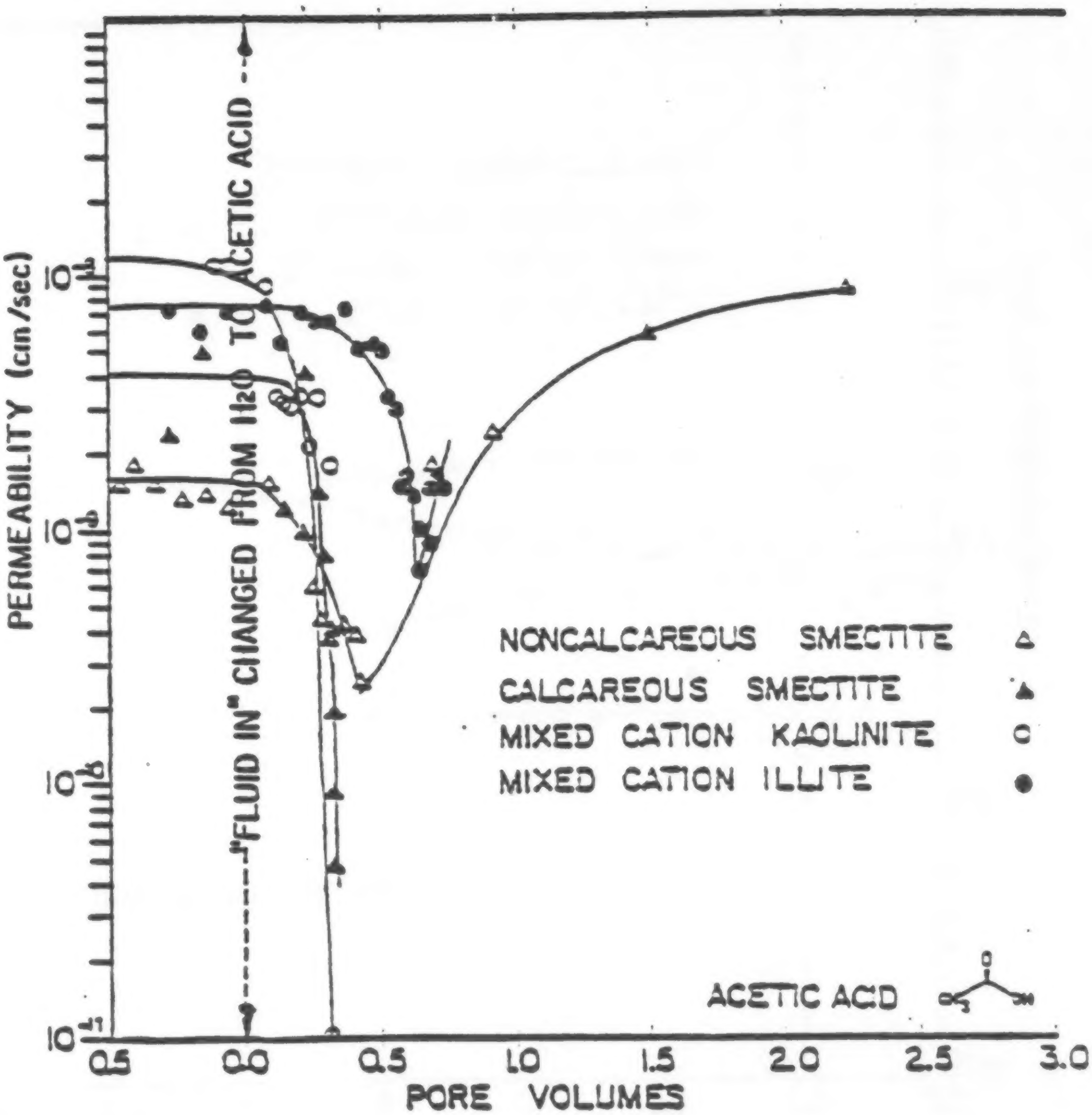
Baseline permeability (two pore volumes) for the four soils is depicted in Figure 4-5. Permeability data obtained for the various organics tested are presented in Figures 4-6 through 4-13 (Anderson, 1981; Anderson et al., 1981; Brown and Anderson, 1982). Findings as described by the authors are excerpted below:

> Acetic Acid (glacial)--All four clay soils permeated with acetic acid showed initial decreases in permeability . . . thought to be due to partial dissolution and subsequent migration of soil particles. These migrating particle fragments could lodge in the fluid-conducting pores, thus decreasing cross-sectional area available for fluid flow.



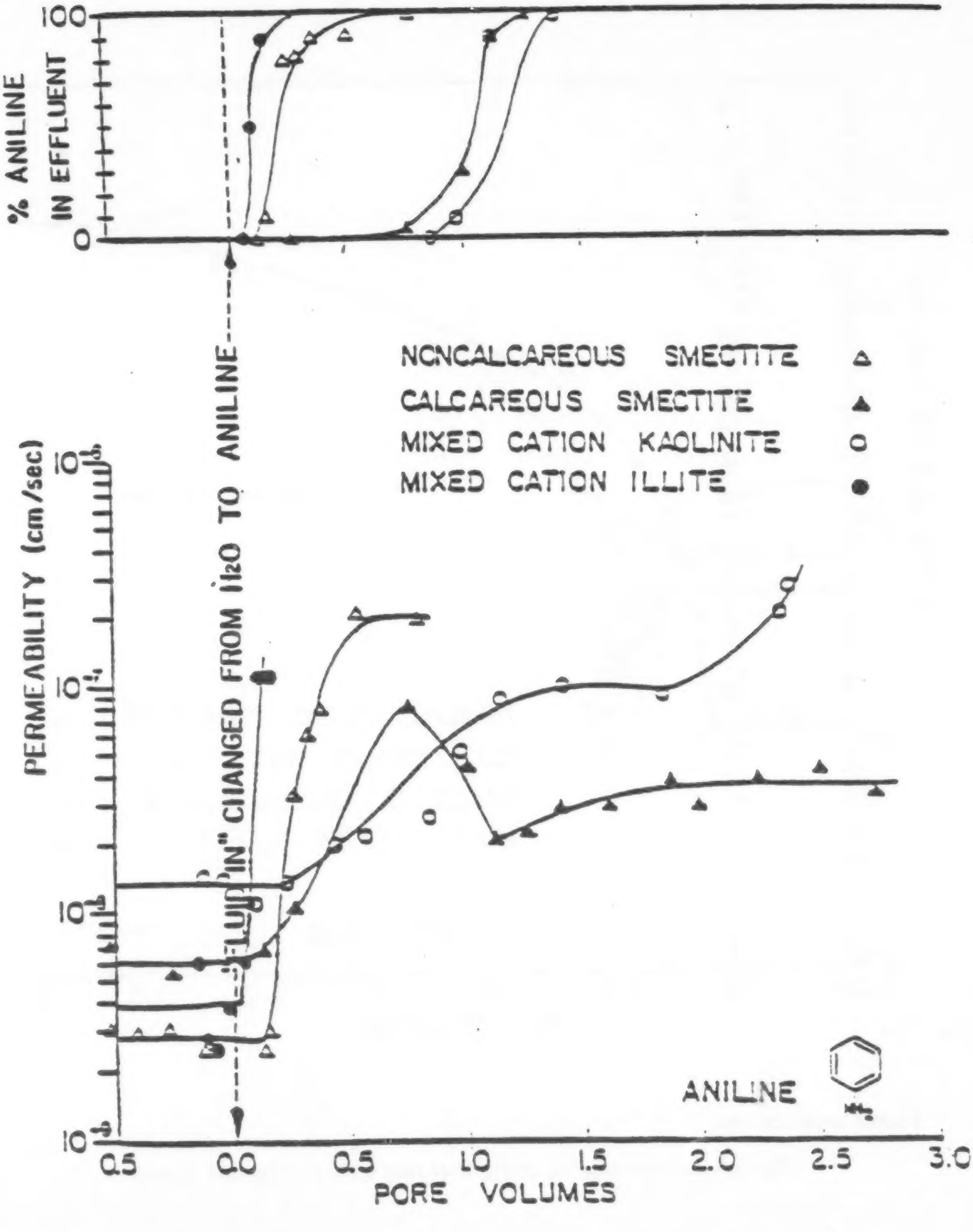
Source: Anderson, 1981

Figure 4-5. Permeability of the four clay soils to water (0.01N $CaSo_3$).



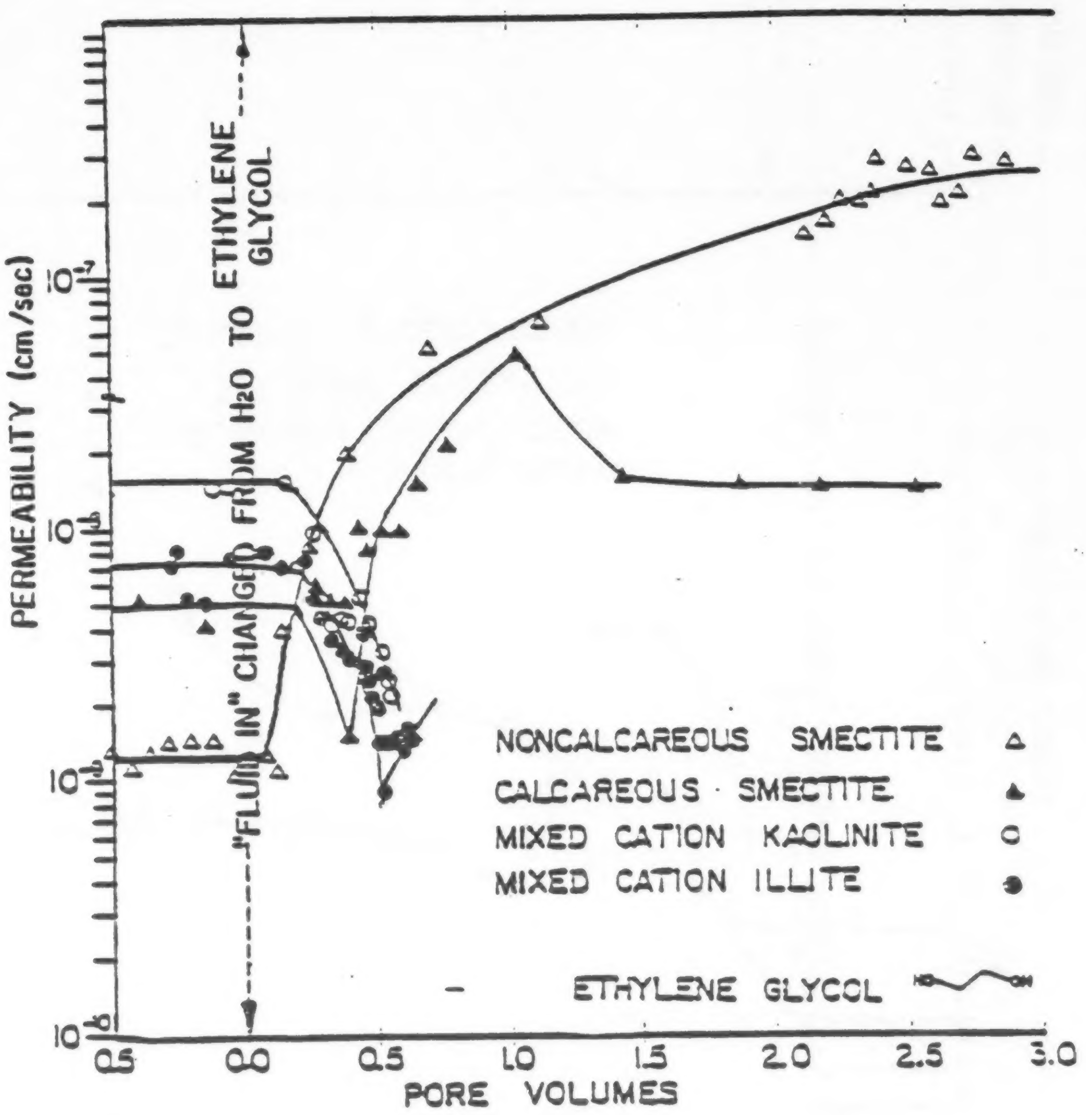
Source: Anderson, 1981

Figure 4-6. Permeability of the four clay soils to acetic acid.



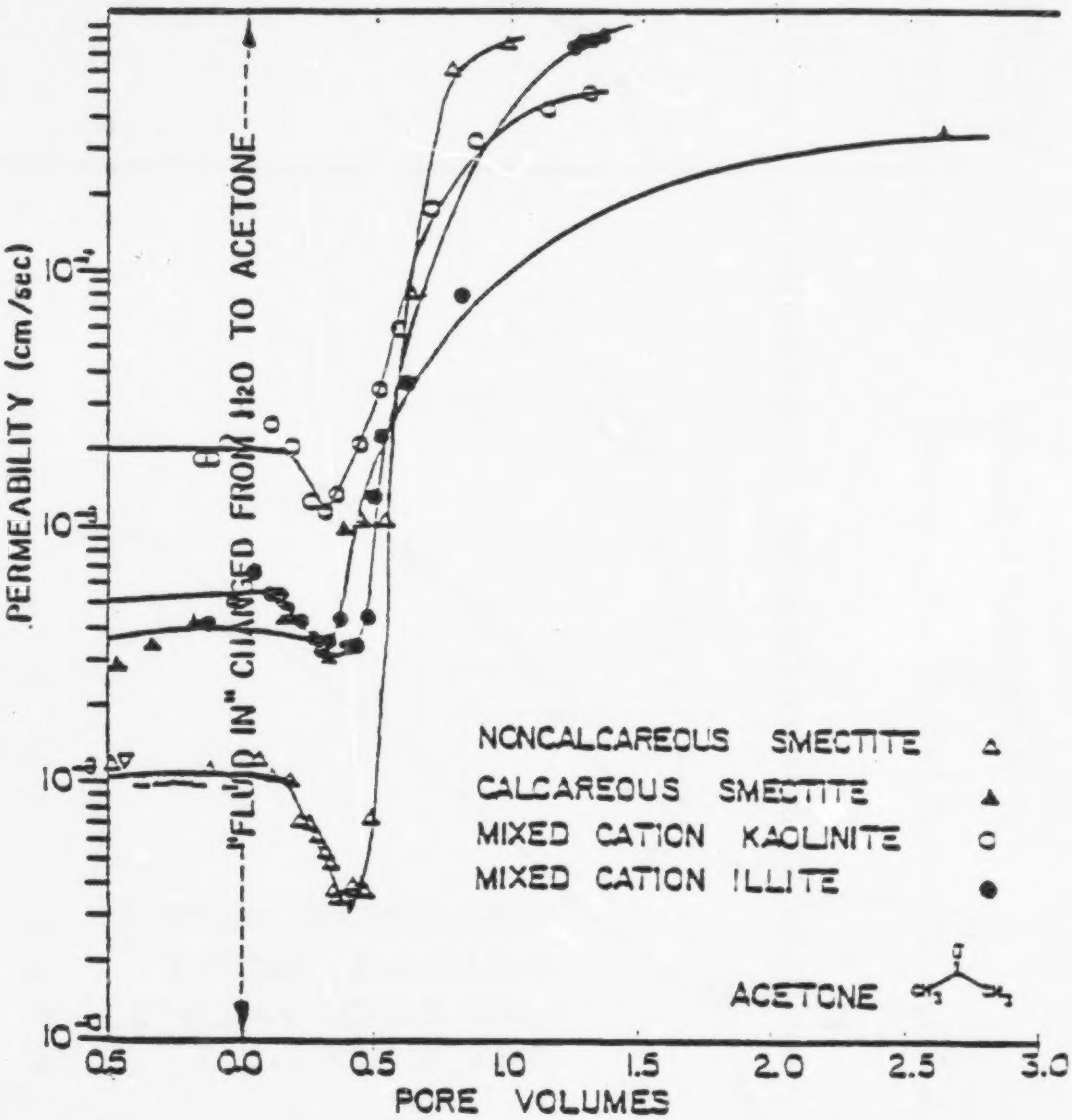
Source: Anderson, 1981

Figure 4-7. Permeability and breakthrough curves of the four clay soils treated with aniline.



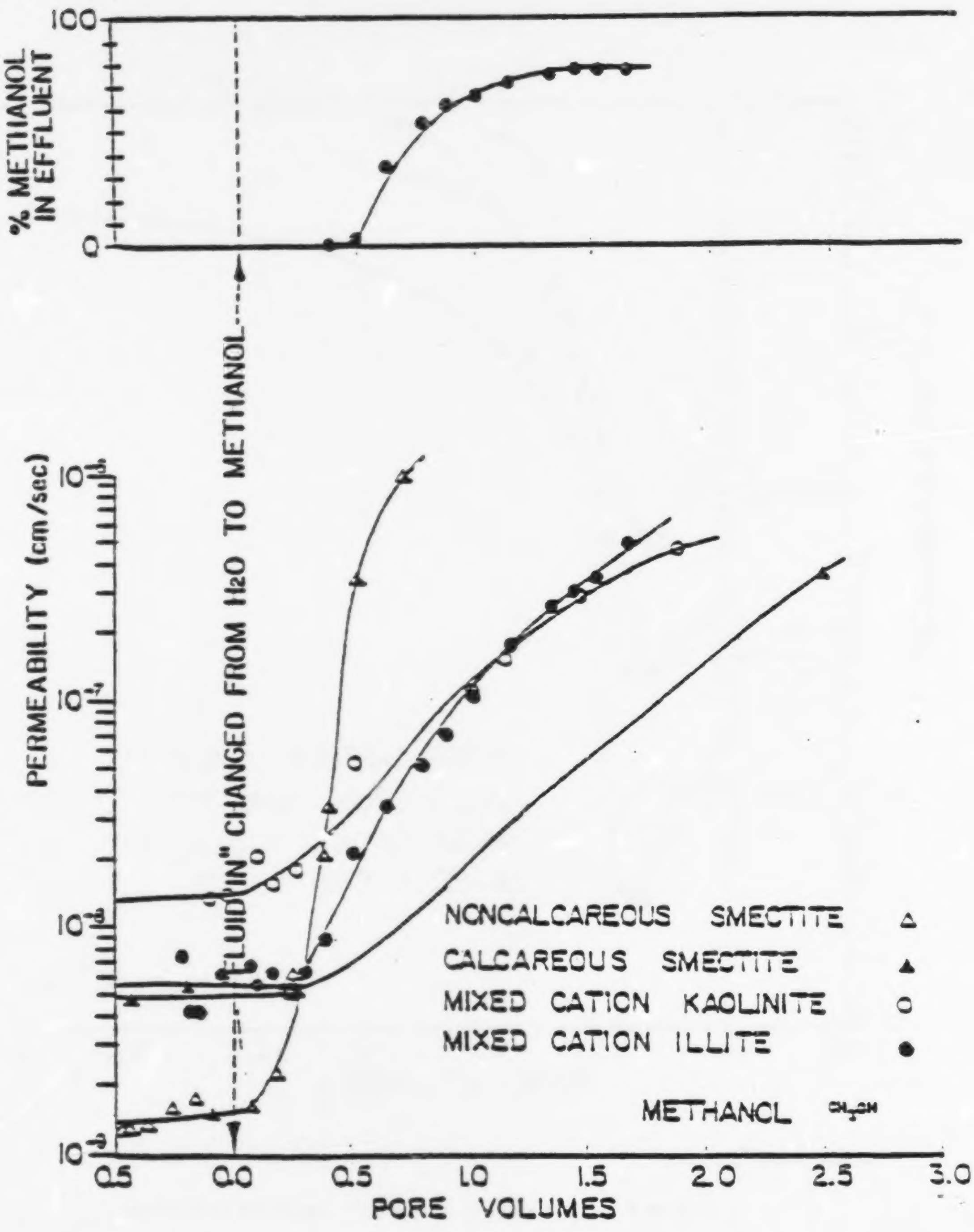
Source: Anderson, 1981

Figure 4-8. Permeability of the four clay soils to ethylene glycol.



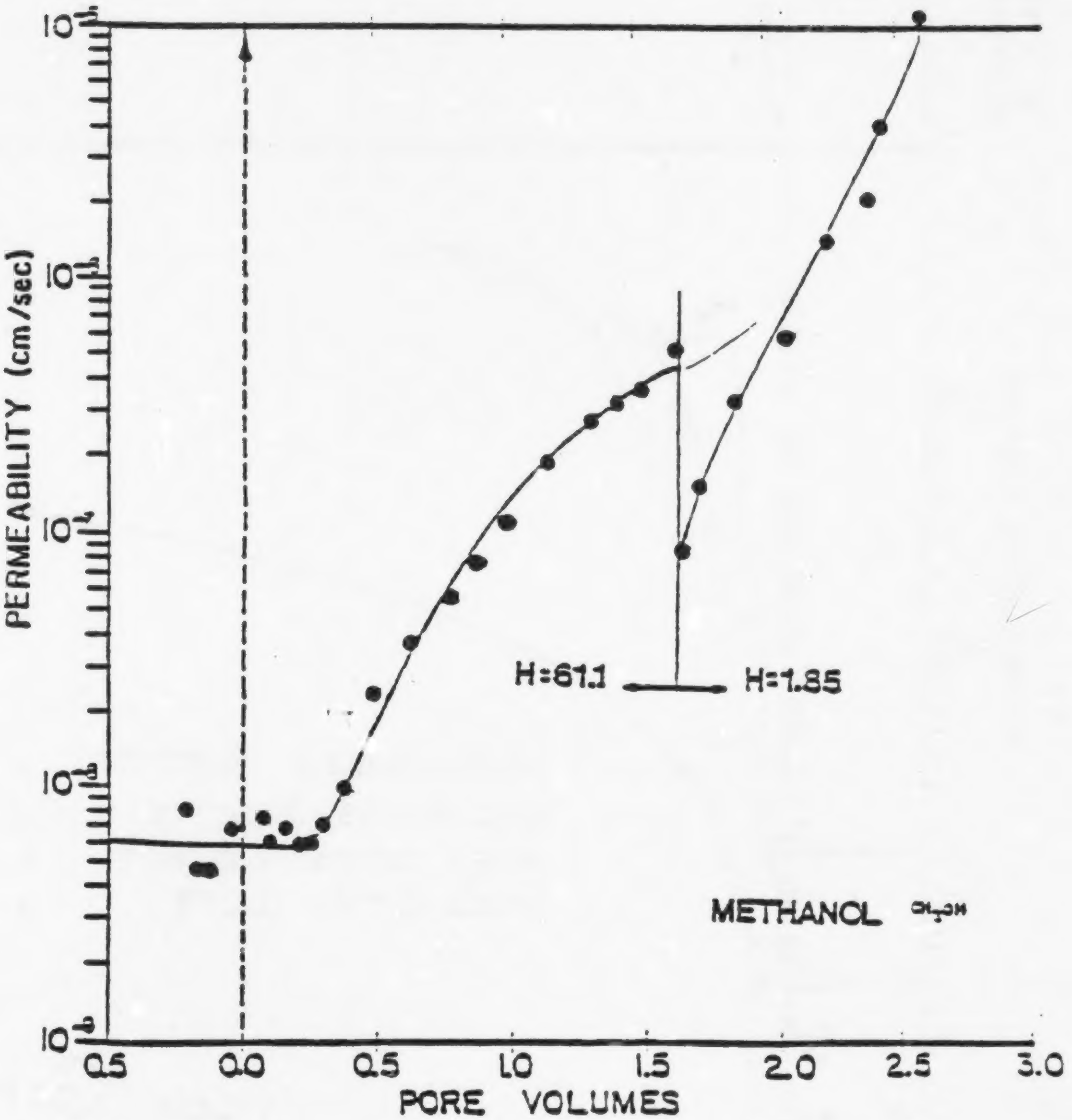
Source: Anderson, 1981.

Figure 4-9. Permeability of the four clay soils to acetone.



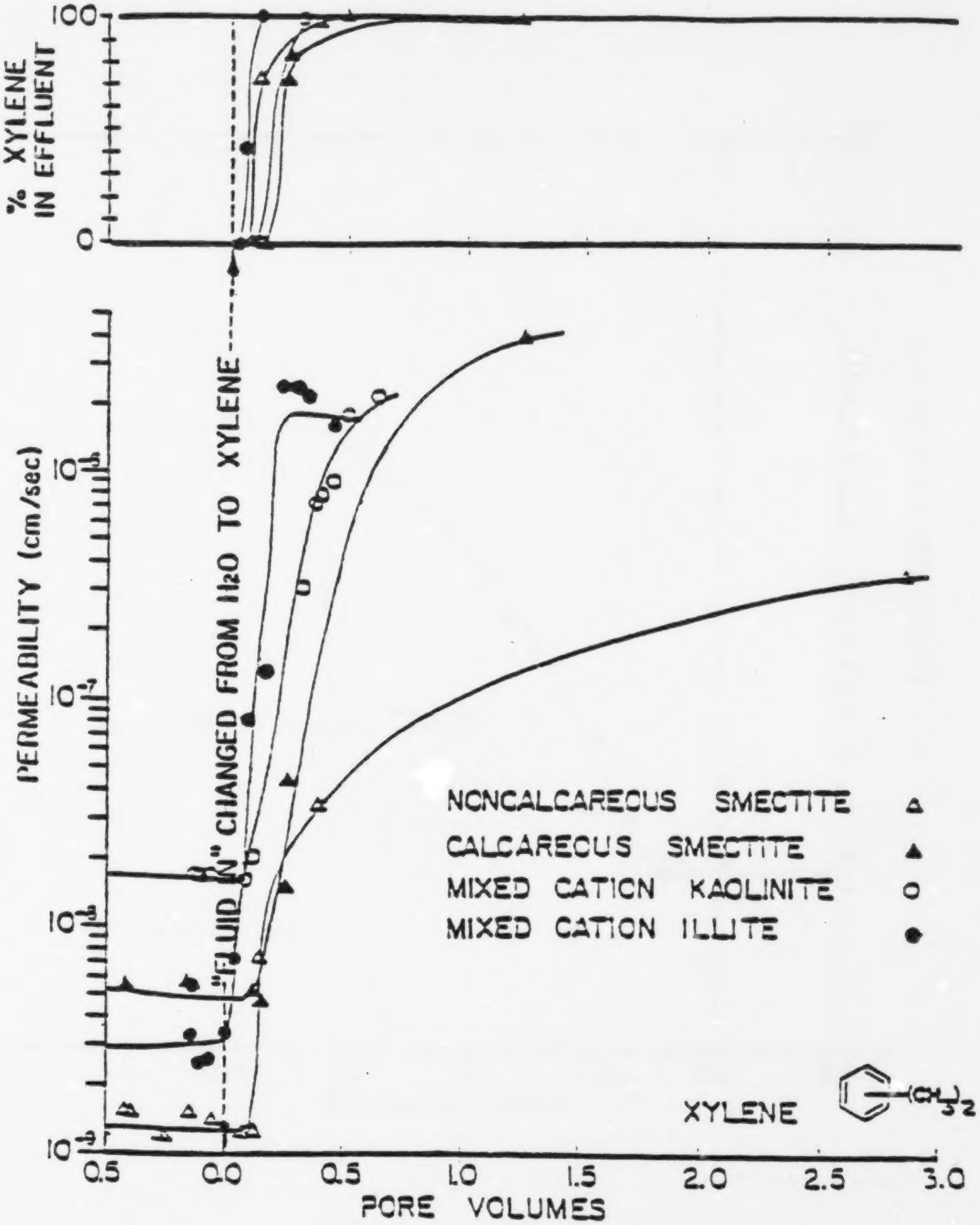
Source: Anderson, 1981

Figure 4-10. Permeability of the four clay soils to methanol and the breakthrough curve for the methanol-treated mixed cation illitic clay soil.



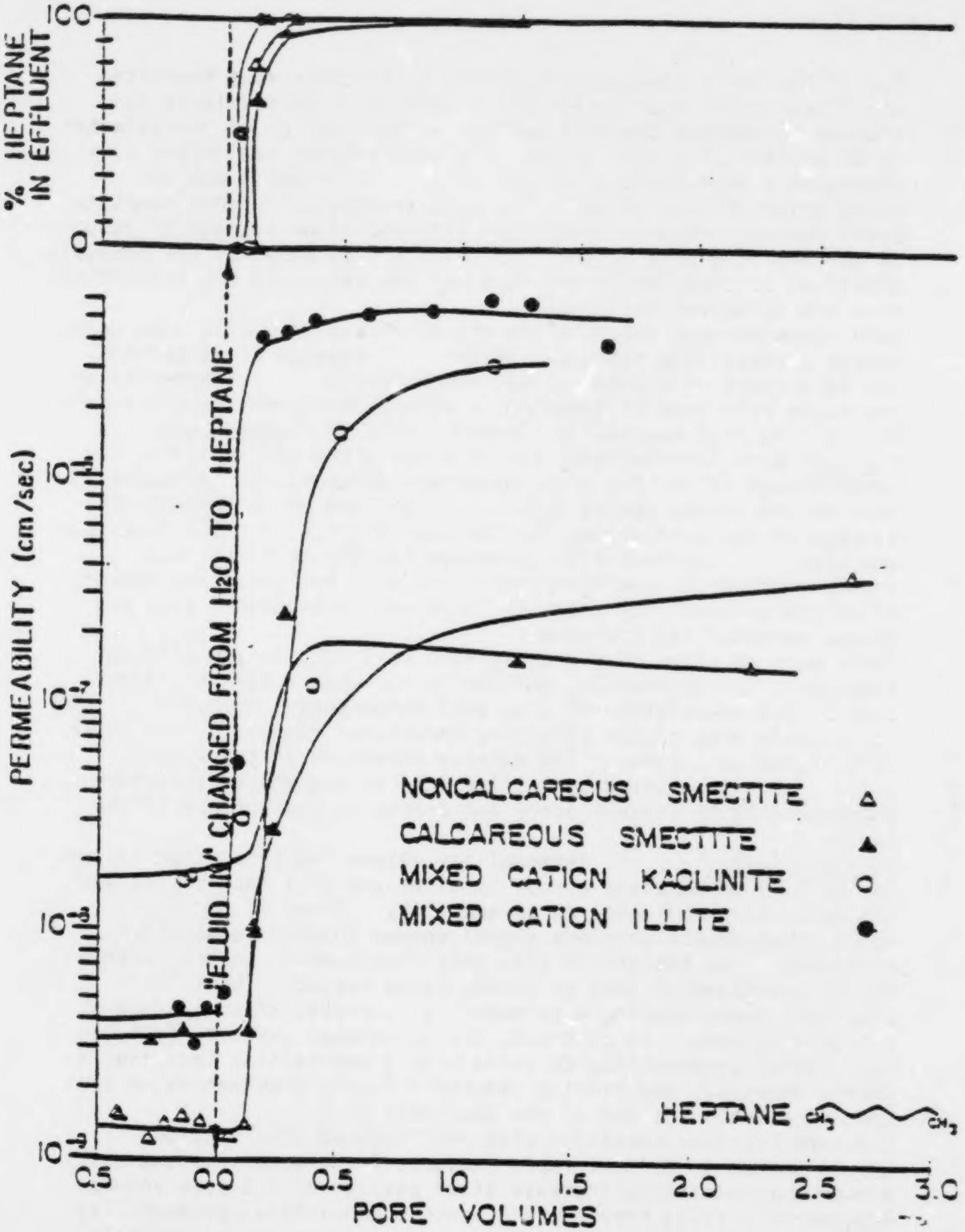
Source: Anderson, 1981

Figure 4-11. Permeability of the methanol-treated mixed cation illitic clay soil at two hydraulic gradients.



Source: Anderson, 1981

Figure 4-12. Permeability and breakthrough curves of the four clay soils treated with xylene.



Source: Anderson, 1981

Figure 4-13. Permeability and breakthrough curves of the four clay soils treated with heptane.

Two of the soils treated with acetic acid (calcareous smectite and mixed cation kaolinite) showed continuous permeability decreases throughout the test period. After passage of approximately 20 percent of a pore volume, the acid treated kaolinitic clay generated a dark red-colored effluent . . . probably due to dissolution of iron oxides. The acid treated calcareous smectite began passing cream-colored foamy effluent after passage of about 28 percent of a pore volume. . . . The creamy material was probably dissolved calcium, while the foam was the result of CO_2 liberation from the dissolved carbonates.

Both noncalcareous smectite and the mixed cation illite eventually showed permeability increases after . . . passage of 39 percent and 62 percent of a pore volume, respectively. . . . Permeability increases with both of these soils were probably due to progressive soil piping that eventually cleared initially clogged pores.

Aniline--Both noncalcareous smectite and mixed cation illite had breakthrough of aniline with concurrent permeability increases at pore volume values (below 0.5). . . . Aniline broke through after passage of one pore volume for the kaolinitic soil. The calcareous smectite . . . permeability increased rapidly at first, but showed substantial decrease concurrent with aniline breakthrough. After the permeability decrease, this soil exhibited a slow but steady permeability increase.

There were no signs of migrating soil particles in any effluent samples. . . . Apparently, aniline is too weak a base to cause significant dissolution of clay soil components. However, . . . the organic base caused extensive structural changes in the upper half of the soil cores. The massive structure of the four soils . . . was altered by aniline into an aggregated structure characterized by visible pores and cracks in the surface of the soils.

Ethylene Glycol-- . . . Permeability values indicated that it was the ability of ethylene glycol to alter the soil fabric that was the dominating influence on permeability. Three of the clay soils treated with ethylene glycol showed initial permeability decreases. The kaolinitic clay soil continued to undergo permeability decreases as long as it was being tested. The illitic clay soil began showing a permeability increase after passage of 0.5 pore volumes. In contrast, the calcareous smectite followed its initial permeability decrease with a substantial increase, a second decrease, and finally reached a nearly constant value that continued until the end of the test period.

The noncalcareous smectitic clay soil treated with ethylene glycol showed an initial rapid increase in permeability and a slower but continuous increase after passage of 0.5 pore volume.

Acetone--All soils treated with acetone had initial permeability decreases. These decreases continued until passage of approximately 0.5 pore volume. During passage of the next 0.5 pore volume, however, the soils underwent large permeability increases. One possible explanation for this sequence of permeability changes is as follows:

1. The higher dipole moment of acetone caused initial increase in interlayer spacing between adjacent clay particles as compared to water only.
2. As more acetone passed through the soil cores, more water layers were removed from clay surfaces. Due to its larger molecular weight, however, fewer acetone layers were adsorbed than had adsorbed when water was the only fluid present. This resulted in a larger effective cross-sectional area available for fluid flow.

While acetone can displace water from clay surfaces due to its higher dipole moment, it cannot form as many adsorbed fluid layers as water due to its higher molecular weight.

Examination of the soil after acetone treatment showed extensive shrinkage and cracking. Such soil shrinkage is usually associated with dehydration, indicating that acetone had extracted water from soil particle surfaces.

Methanol--Unlike soils treated with acetone, methanol-treated soils underwent no initial permeability decrease.

Percent methanol in the effluent from the illitic clay soil paralleled an increase in permeability of the soil. After passage of 1.5 pore volumes, the hydraulic gradient was reduced from 61.1 to 1.85 and another pore volume of methanol passed. After an initial decrease, permeability of the soil steadily increased at the lower hydraulic gradient. . . . No particle migration was detected in effluent from methanol-treated cores, and therefore soil piping was discounted as a mechanism for observed permeability increases. Examination of methanol-treated soil cores revealed development of large pores and cracks visible on the soil surface. The lower dielectric constant of methanol may have caused a decrease in interlayer spacing of the clay minerals present in the soils and thereby promoted the structural changes.

Xylene--Xylene-treated soils showed rapid permeability increases followed by nearly constant permeabilities roughly two orders of magnitude greater than their permeabilities to water. Permeability increases may be caused by . . . structural changes in the xylene-treated soils, exemplified by massive structure before treatment and blocky structure after the soils were treated with xylene.

Heptane--Permeability patterns for the heptane cores closely approximated those shown by the xylene-treated cores (i.e., large initial permeability increases). Following these initial large increases, rate of permeability increase slowed until nearly constant permeability values were observed.

Only the calcareous smectitic clay showed . . . permeability to heptane well below its permeability to xylene. The constant permeability values eventually reached by the neutral nonpolar treated cores were probably related to the limited ability of these fluids to penetrate interlayer spaces of the clay minerals.

4.5.9.3 Reintroduction of Water--

When the standard calcium sulfate solution was reintroduced to the noncalcareous smectite test columns permeated with aniline, methanol, and ethylene glycol, a subsequent decrease in the permeability (roughly 1 order of magnitude) was observed. The final permeability (after one pore volume of standard solution) remained well above the baseline permeability measured prior to introduction of the organic solvents (Anderson, 1981).

4.5.9.4 Discussion--

Viscosity and density differences between the organic fluid and water do not account for the large changes in permeability observed in the tests. The data from these permeability tests illustrate the importance of passing multiple pore volumes of fluid through the soil samples in order to determine the effects of solvents on permeability.

The work by Anderson has been criticized as being "unrepresentative of real conditions" because pure organic solvents (rather than waste leachate) were used in the permeability tests (Gray and Stoll, 1983). The test results stand, however, as a demonstration of potential effects on clay liners should they come in contact with certain types of undiluted organic liquids.

The high gradients used in the tests have also been questioned (Zoeller, 1982; Gray and Stoll, 1983) since it is unlikely that gradients as high as 361 would be encountered in the field. The use of elevated gradients in laboratory testing in order to reduce testing time is not uncommon. Although the gradients used by Brown and Anderson were higher than those used by most researchers, the permeability changes observed cannot be attributed to the elevated gradients. Conditions were maintained well within the laminar flow regime (i.e., Reynolds number was below 10) so that Darcy's Law for computing the permeabilities remained valid. In general, the trends observed in these tests are consistent with data from other researchers, and explanations can be linked to the findings from clay-chemical complex studies.

Complete saturation in the compaction permeameter without backpressure cannot be ensured. It is possible that air introduced with the permeant fluids could have influenced the test results to some extent, but probably by no more than one order of magnitude.

Finally, the use of fixed-wall permeameters for these studies has been criticized (because of the potential for sidewall leakage) by advocates of triaxial test methods. As there are advantages and disadvantages with either method, the argument as to which test is most meaningful is finally a matter of opinion.

Acetic acid, a weak organic acid, has a dissociation constant of 1.75×10^{-5} in aqueous solution (pKa = 4.75) at 25 °C. Since glacial acetic acid was used, the extent of ionization was a fraction of this. The permeability decreases observed probably would have been followed by sharp increases in permeability due to dissolution of the basic soil components if a dilution had been used rather than glacial acetic acid. Progressive soil piping, as pointed out by Brown and Anderson, follows the soil dissolution and eventually results in large pores with an associated increase in permeability.

This effect (i.e., increased permeability) is to be expected with carboxylic acids or sulfonic acids in general. This is of particular significance since carboxylic acids are among the biological degradation products of a wide range of organic compounds. Soils that are high in carbonates are most affected by acids (even weak acids such as acetic acid), but other soil constituents such as Fe_2O_3 are also susceptible.

4.5.10 Schramm's Study (1981) of the Permeability of Soil to Organic Solvents

Schramm (1981) evaluated permeabilities for eight soils tested with kerosene, isopropyl alcohol, ethylene glycol, and mixed xylenes. Permeability with water was also measured. A total of 211 column tests were run to determine saturated permeabilities to the various test fluids. The research was performed at the University of Arizona.

Clay content in the soils ranged from 1 percent in River Bottom sand to 70 percent in Lake Bottom clay. Characteristics of the soils are presented in Table 4-10.

4.5.10.1 Test Method--

Permeabilities were obtained in glass or plexiglass cylinders through the falling head method. Cylinders 5.8 cm in diameter were used in the tests with soil packed to a depth of 5 cm. Soil was retained in the cylinders by a galvanized screen glued to the cylinder. Prior to testing, samples were immersed overnight in the test solvent. During the test, the height of the solvent column above the soil surface varied between 6 and 2 cm. Six replicates were tested for every solvent-soil combination. Experiments were repeated until a constant permeability value was obtained. Length of runs varied from 1-1/2 minutes to several weeks. An analysis of variance was performed to evaluate differences between intrinsic permeability for the various soils and solvents.

4.5.10.2 Test Results--

A summary of the saturated permeability values (expressed in cm/h) obtained from these tests is given in Table 4-11. It is notable that with each soil tested the highest permeabilities were obtained with the nonpolar organics, kerosene, and xylene. (Kerosene is a mixture of C_{12}-C_{18} compounds and includes both aliphatics and aromatics.)

Intrinsic permeabilities were also calculated to take into account differences in viscosity and density for the various permeant fluids. Analysis of the intrinsic permeability values included a one-way variance test to determine if the intrinsic permeability varies significantly with solvents or with soils. The intrinsic permeabilities for a given soil are expected to be constant unless there is interaction between the soil and the permeating liquid. The differences in intrinsic permeability with the

TABLE 4-10. CHARACTERISTICS OF SOILS USED IN PERMEABILITY TESTS[a]

Soil name	Soil order	Clay %	Silt %	Sand %	Soil paste pH	Cation exchange capacity (meq/100 g)	Electrical conductivity of saturated extract (μmho/cm)	Column bulk density (g/cm^3)	Soil surface area (m^2/g)	Predominant clay minerals
Lake Bottom clay	Entisol	70.6	24.0	5.4	7.7	34.7	1,111	1.52	142.0	Illite Kaolinite
Nicholson	Alfisol	49.0	47.0	3.0	6.7	37.0	176	1.53	120.5	Vermiculite
Fanno	Alfisol	46.0	19.0	35.0	7.0	33.0	392	1.48	122.1	Montmorillonite Mica
Chalmers	Mollisol	31.0	52.0	14.0	6.6	22.0	288	1.53	95.6	Montmorillonite Vermiculite
Canelo	Alfisol	28.0	28.6	43.4	5.6	5.76	240	1.72	35.0	
Anthony	Entisol	15.0	14.0	71.0	7.8	10.0	328	1.87	49.8	Montmorillonite Mica
Mohave	Aridisol	11.1	37.0	52.0	7.3	10.0	615	1.78	38.3	Mica Kaolinite
River Bottom sand	Entisol	1.0	2.0	97.0	7.2	2.0	210	1.80	3.6	Kaolinite Mica

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[a] Adapted from Schramm (1981).

TABLE 4-11. PERMEABILITY COEFFICIENTS (cm/s) DETERMINED IN SOILS TESTED WITH ORGANIC SOLVENTS[a]

Soil	Solvent				
	Water	Kerosene	Isopropyl alcohol	Ethylene glycol	Xylene
Lake Bottom clay	5.0×10^{-5}	3.4×10^{-4}	2.1×10^{-4}	1.1×10^{-5}	1.7×10^{-3}
Nicholson	4.2×10^{-6}	9.5×10^{-5}	2.3×10^{-5}	6.8×10^{-7}	3.5×10^{-4}
Fanno	1.5×10^{-4}	5.2×10^{-3}	3.1×10^{-3}	3.0×10^{-4}	1.8×10^{-2}
Chalmers	1.8×10^{-5}	3.8×10^{-5}	2.3×10^{-5}	1.8×10^{-6}	1.7×10^{-4}
Canelo	5.0×10^{-6}	5.6×10^{-5}	3.5×10^{-5}	1.4×10^{-6}	1.5×10^{-4}
Anthony	9.9×10^{-5}	2.0×10^{-4}	9.5×10^{-5}	5.8×10^{-6}	4.4×10^{-4}
Mohave	1.9×10^{-3}	2.6×10^{-3}	1.6×10^{-3}	2.0×10^{-4}	8.5×10^{-3}
River Bottom sand	1.7×10^{-2}	3.4×10^{-2}	5.8×10^{-3}	6.4×10^{-4}	1.8×10^{-2}

[a] Data from Schramm (1981).

different solvents are shown in Figure 4-14 for five soils. The trend for increased intrinsic permeability is readily apparent.

The analysis also shows rather uniform behavior of intrinsic permeability for solvents in the different soil types, the Fanno, Mohave, and River Bottom Sand having highest relative intrinsic permeabilities with all organic solvents.

From the data summary and analysis of variance, Schramm (1981) concluded that:

> Calculated values of intrinsic permeability vary for the same soil depending on solvent. However, the variation is relatively minor compared to variation due to differences in soil properties among several soils. . . .
> Since intrinsic permeability values vary for the same soil depending on solvent, the soil and solvents are interacting.

4.5.10.3 Discussion--

The test method used by Schramm differed significantly from that employed by Anderson (1981) to measure permeability. Notable differences include the following:

- Gradients imposed on the sample were very low.
- No estimate was made of the pore volumes passed through the test samples.
- Samples were presaturated with the test fluid rather than a standard calcium-sulfate solution.

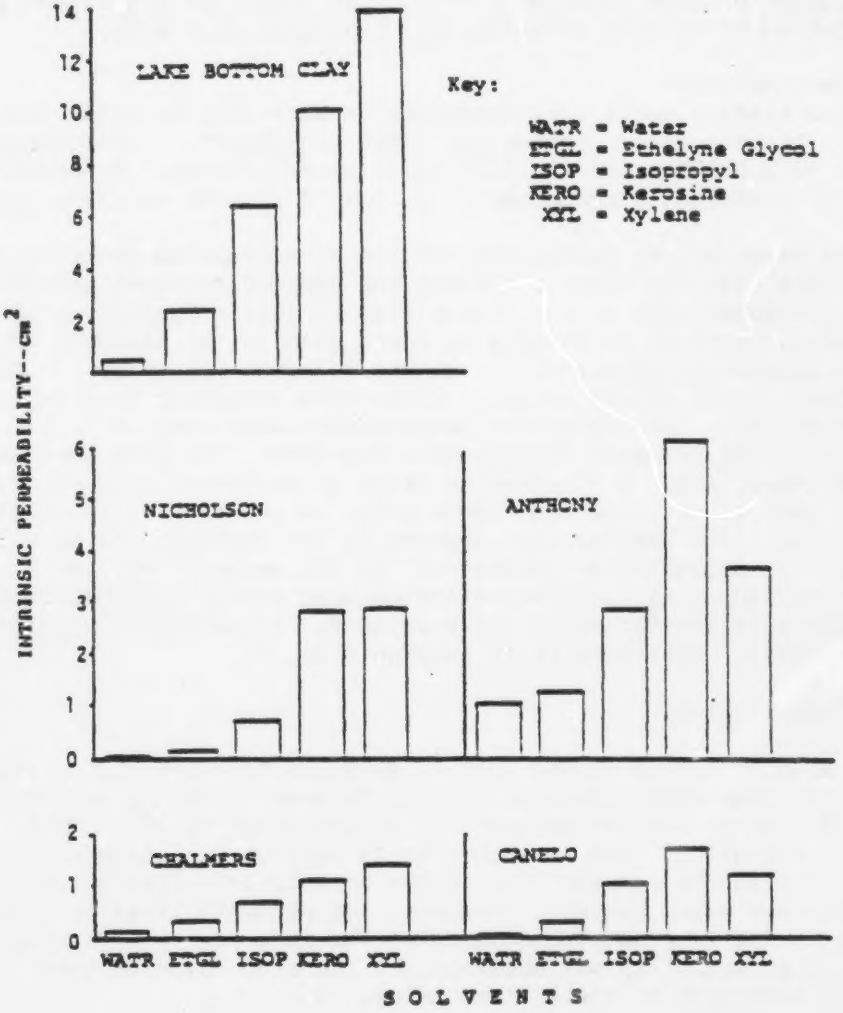
The impact of the low hydraulic gradient on the test results is not known. The concept of "threshold gradient" has been postulated, although there is no evidence of this effect in the results reported by Schramm.

Although Schramm reports that tests were continued until constant permeability values were obtained, the criterion to establish an acceptable difference in consecutive measurements was not discussed. There is no indication of the number of pore volumes exchanged.

It may be argued that saturating the samples using the test permeant fluid rather than water or a standard permeant solution is representative of conditions typical for a liner in service in a land disposal application.

4.5.11 Evaluation by Monserrate (1982) of the Permeability of Two Clays to Selected Electroplating Wastes

Monserrate (1982) at Duke University investigated the permeability of two clays exposed to simulated electroplating wastes. The purpose of the research was to examine the effect of compaction on the permeability response of the two clays. The clays tested were a Wyoming bentonite and a White



Source: Schramm, 1981

Figure 4-14. Variation of intrinsic permeability with solvent for each soil.

Store clay from North Carolina. The White Store clay was reported to be an active clay.

The simulated electroplating wastes used as permeant fluids were a 1-molar solution of zinc chloride ($ZnCl_2$) (136.3 g/L, pH = 5.5) and a 1-molar solution of chromic acid (H_2CrO_4) (100 g/L, pH = 1.5).

4.5.11.1 Test Method--

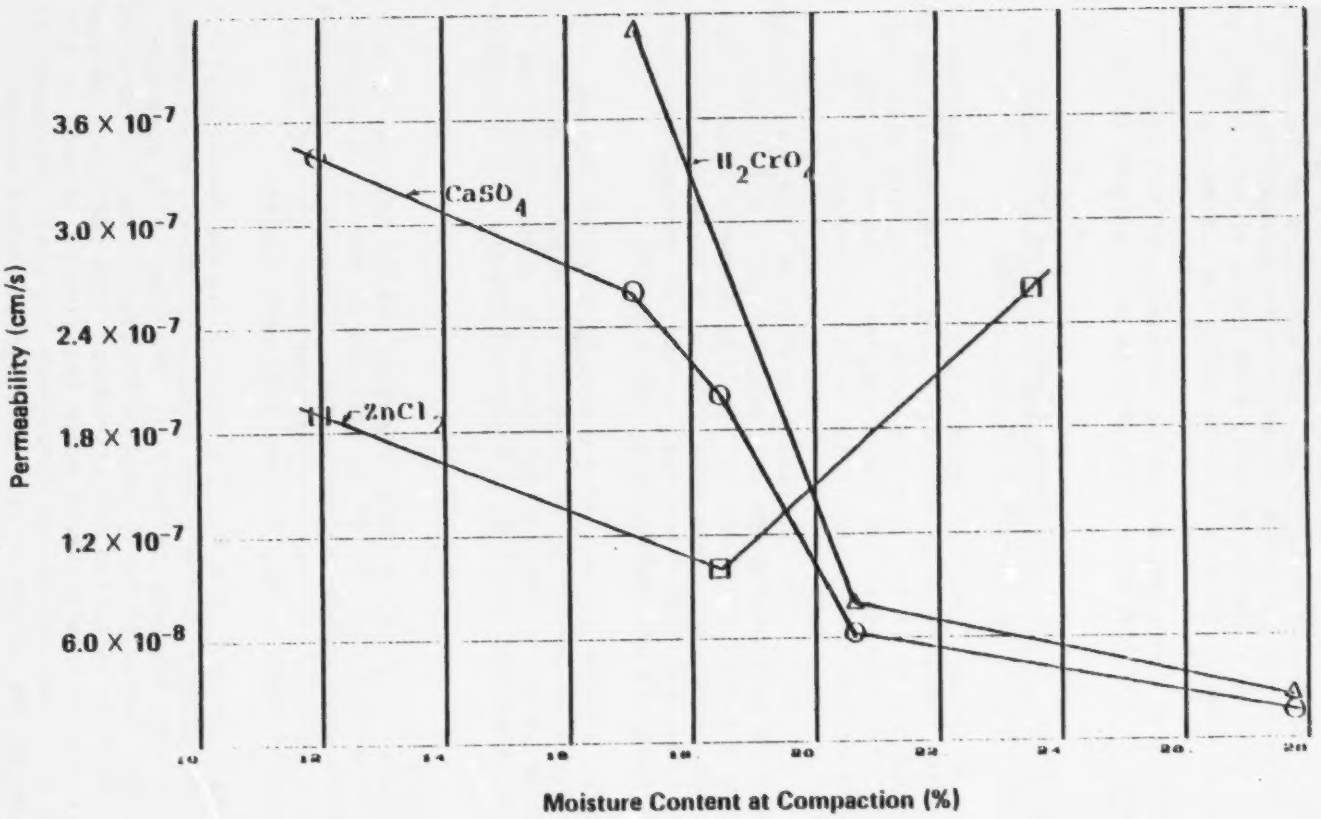
Standard Proctor tests were conducted on each clay to establish the relationship between water content and compacted density. Procedures outlined in ASTM D-698 Method A (ASTM 1985) were followed. Permeability studies were subsequently conducted at several different moisture contents.

For the permeability tests, the samples were prepared by mixing 0.01 N calcium sulfate with the clay to achieve the desired moisture content. The clays were compacted into either zinc-plated or lucite compaction permeameters using compactive efforts comparable to those used in the standard Procter tests. The compaction permeameters measure 4 inches (10.16 cm) in diameter and 4.6 inches (11.68 cm) in height. Clays were compacted to a depth of 2 inches (5.08 cm). The compaction permeameters were adapted to allow measurement of flow as the permeant fluid enters the unit. The data were evaluated as a falling head test. A standard solution of deaired 0.01 N calcium sulfate was used to saturate the sample prior to introducing the test permeant fluids. The samples were exposed to the permeant fluids under approximately 75 pounds of air pressure. In the permeability tests, a statistical procedure was used to determine when enough data had been accumulated to make a determination of the k value for a certain clay-permeant fluid combination. (See details in Section 4.3).

4.5.11.2 Test Results--

Tests with bentonite showed that at moisture contents in the proximity of the optimal compacted moisture content, permeabilities to calcium sulfate, chromic acid, and zinc chloride were all on the order of 10^{-10} cm/s. At lower moisture contents, the permeability to calcium sulfate was higher by a factor of 2 than the permeability to the chromic acid test solution. At wet-of-optimum water contents, however, the permeabilities determined for the two fluids were nearly identical. The same low water content divergence in permeability was observed for the zinc chloride test fluid, although the magnitude of the difference was less.

The permeability of White Store clay to standard calcium sulfate solution and to the test fluids is shown in Figure 4-15 as a function of the moisture content at compaction. The White Store clay appeared to be more sensitive than the bentonite clay to changes in compacted moisture content. The permeability to the chromic acid test fluid aligns closely with that of calcium sulfate at moisture contents higher than about 20 percent (optimum moisture content = 21.5 percent). At lower than about 20 percent moisture, the permeability to the chromic acid was higher than to calcium sulfate. Zinc chloride test fluid showed permeabilities lower than for calcium sulfate at low moisture.



Source: Monseratte, 1982

Figure 4-15. Permeability of White Store clay to 0.01 N calcium sulfate, chromic acid (1 molar), and zinc chloride (1 molar) as a function of moisture at compaction.

Monserrate (1982) concluded that for both bentonite and White Store clays, the application of zinc chloride and chromic acid did not alter the clay's permeability response, indicating that the structure of the clay, as compacted, is the predominant influence in determining permeability. She noted that the test results differed markedly from those of Coia (1981) who carried out similar studies at Duke University using the chromic acid.

Monserrate noted that there was extensive corrosion of the steel permeameters and some of the pressure fittings that were exposed to the chromic acid test fluid.

4.5.12 Research by Brown, Green, and Thomas (1983) on the Effect of Two Organic Hazardous Wastes on Simulated Clay Liners

Brown, Green, and Thomas (1983) at Texas A & M University reported results of permeability tests in which clay soil samples representative of three types of simulated clay liners were subjected to a xylene waste and an acetone waste in both laboratory and field studies. The three clay materials tested were predominantly kaolinite, mica, or bentonite. The soils were blended with sandy loam soil to attain permeabilities in the range of 1×10^{-7} to 1×10^{-8}. The laboratory studies in fixed-wall permeameters were directed at determining the influence of different initial moisture contents and elevated gradients on the clay permeabilities. The procedures used in the laboratory tests were similar to those described by Anderson (1981) and Brown and Anderson (1982) in earlier permeability tests with organics. The research was sponsored by the U.S. EPA.

The xylene waste, a paint solvent that was used to clean factory sprayer lines, contained 25 percent paint pigments and trace amounts of water. The acetone waste was a chemical manufacturing waste containing 91.7 percent acetone, 4 percent benzene, 0.6 percent phenol, and 3.7 percent unknown.

4.5.12.1 Test Methods--

For the laboratory tests, soils were compacted wet of optimum to at least 90 percent Proctor density. Prior to exposure to the waste solvents, samples were saturated with 0.01 N calcium sulfate. Unsaturated cores were also subjected to the permeability tests. Pressures of 5, 15, and 30 psi (equivalent to hydraulic heads of 31, 91, and 181) were tested.

Twenty-eight field cells (1.5 m × 1.5 m × 1.8 m inside dimensions) were constructed of reinforced concrete and lined with 100-mil high-density polyethylene to facilitate leachate collection. The clay soils were compacted with a vibratory compactor to 95 percent Proctor in two 7.5-cm-thick lifts. The liquid waste to be tested was introduced into each cell through a standpipe into barrels that were placed above the liners. A head of 1 m of liquid (gradient of 7) was maintained throughout the test. Permeabilities were calculated based on the volume of cell drainage collected during discrete time intervals. Following the permeability tests, each cell was disassembled and subjected to chemical and morphological analysis. The wastes to be tested in the field cells were dyed with Automate Red B and Fluorescent Yellow to facilitate detection within the sample cores.

4.5.12.2 Laboratory Test Results--

To compare the results from the various tests, calculated permeabilities were plotted as a function of the number of pore volumes of effluent that passed through the sample. Permeabilities to standard calcium sulfate solution on similar soils in the laboratory were generally found to be reproducible to within 0.25 order of magnitude. Permeabilities increased, however, when the organic-solvent wastes were substituted as the permeant fluids. Permeability to the organic-solvent wastes was typically 2 to 4 orders of magnitude greater than the permeabilities measured with the calcium sulfate solution. Presaturation with 0.01 N calcium sulfate appeared to retard the effect of the permeant fluids on the samples (i.e., more pore volumes were displaced before breakthrough occurred in the presaturated samples). Highest permeabilities measured on initially unsaturated samples were typically 1 to 2 orders of magnitude greater than for samples that were initially saturated.

Permeability of the clay soils increased rapidly upon exposure to xylene waste after the cumulative flow exceeded 0.2 to 0.4 pore volume. The behavior of acetone was characterized by an initial small decrease in permeability (minimum at approximately 0.5 pore volume), which was followed by a steady increase in permeability. This behavior was explained as follows: acetone first caused swelling of the clay soil with low concentrations initially diffusing into the pores; as more of the water was displaced by the solvent, shrinkage occurred.

Similar increases in permeability were observed for all three soil types. The increase in permeability, however, for the bentonite samples occurred after a larger pore volume was displaced.

If the permeability values calculated from the test data are plotted as a function of pore volumes, the different hydraulic gradients used in the experiments (31, 91, 181) have little influence on either the increase in permeabilities observed or on the final values achieved. (This conclusion holds for both presaturated and unsaturated samples [Brown et al., 1984]).

4.5.12.3 Field Test Results--

In the field studies, xylene had penetrated 10 of the 12 clay liners within 12 months after installation; acetone had penetrated 2 of the liners but at a slower rate. Increases in permeability of the bentonite clay soil to xylene were more dramatic (as much as 2 orders of magnitude increase) than of the other soils and occurred after passage of 0.5 pore volume of permeant fluid. Kaolinitic and micaceous soils showed immediate increases in permeability to xylene. Permeability values continued to increase as more pore volumes of fluid passed through the sample. Increases of more than 2 orders of magnitude were observed in some samples after passage of two pore volumes of xylene.

Field tests with the acetone waste exposed to micaceous clay soil showed a permeability decrease with the first 0.5 pore volume. This was followed, however, by an increase in permeability. After two pore volumes had passed through the liner, the permeability was slightly higher than the initial value measured.

Visual inspection of the clay liners following permeation by the liquid wastes was facilitated by the dyes and by the paint pigment in the xylene waste. Dyed surfaces observed throughout the liner indicated that the liquid had moved through the soil through preferential channels rather than in a clearly defined wetting front. Preferential channels were occasionally found to extend from the top to the bottom of the liner. Evidence that xylene moved through cracks in the liner or around blocky subangular structural components was also obtained through chemical analysis of sections of the permeated liners. The channels observed in the clays are attributed to the displacement of water by the chemicals resulting in desiccation that caused the clay to shrink and crack.

4.5.12.4 Discussion--

The results of this study, in close agreement with those previously found by Brown and Anderson (1982), indicate that severe permeability increases can occur when certain clay materials are in contact with concentrated organic solvents. Although the hydraulic heads used in the laboratory test procedures are higher than are likely to be encountered in the field, the data suggest that the permeabilities measured are independent of the hydraulic gradient used in the test. These results further support the conclusions by Anderson based on tests involving a gradient of 361.

The effect of presaturation with calcium sulfate on the highest permeabilities is significant. Laboratory tests by other researchers (Green et al., 1979; Foreman and Daniel, 1984) have also involved introducing the test permeant fluid onto the unsaturated sample.

4.5.13 Study by Brown, Thomas, and Green (1984) of the Effect of Dilutions of Acetone and Mixtures of Xylene and Acetone on Permeability of a Micaceous Soil

To determine the effect of diluting a polar organic solvent with water, Brown, Thomas, and Green (1984) measured the permeability of an unsaturated micaceous compacted soil to several acetone-water dilutions. Samples were tested at a gradient of 91 in laboratory tests with fixed-wall permeameters. Solutions tested were 100, 75, 50, 25, 12.5, and 2 percent acetone. Methods were similar to those used previously by Anderson (1981).

Compared to permeability to water (actually 0.01 N calcium sulfate), an increase in permeability was seen for solutions where the concentration of acetone was 75 percent or 100 percent. Samples permeated with lower concentrations of acetone did not show appreciable changes compared to the permeability to water.

Similar tests were conducted to determine the effect on permeability of mixtures of xylene and acetone (Brown et al., 1984). For 100 percent xylene, the permeability was reported to be 4 orders of magnitude greater than the permeability to water (0.01 N calcium sulfate). When a mixture of

87.5 percent xylene and 12.5 percent acetone was tested, the permeability was dramatically reduced (i.e., more than 3 orders of magnitude below the permeability to pure xylene). Values were comparable for a mixture containing 25 percent acetone and 75 percent xylene and for a 1:1 mixture. When the acetone component was increased to 75 percent, however, the permeability increased to approximately the value obtained in similar tests with pure acetone. This value was about 1.5 orders of magnitude greater than the permeability of the sample to 0.01 N calcium sulfate.

4.5.14 Tests by Brown, Thomas, and Green (1984) to Determine the Permeability of Micaceous Soil to Petroleum Products

Brown, Thomas, and Green (1984) also reported permeability tests with five commercial petroleum products. A compacted micaceous soil, unsaturated, was permeated with the test fluids at a gradient of 91. The results of the tests with kerosene, diesel fuel, paraffin oil, gasoline, and motor oil are shown in Figures 4-16 through 4-20, respectively. Conclusions of the tests show that the micaceous soil is more permeable to these petroleum products than to water (0.01 N calcium sulfate).

In tests with kerosene, the permeabilities were 3 to 4 orders of magnitude higher than values measured with 0.01 N calcium sulfate in similar samples. One-half the difference was achieved within the first 0.1 pore volume. The permeability to paraffin oil was about 1 order of magnitude greater than the comparable value to the calcium sulfate solution despite its high viscosity. Maximum values were achieved by the passage of one pore volume.

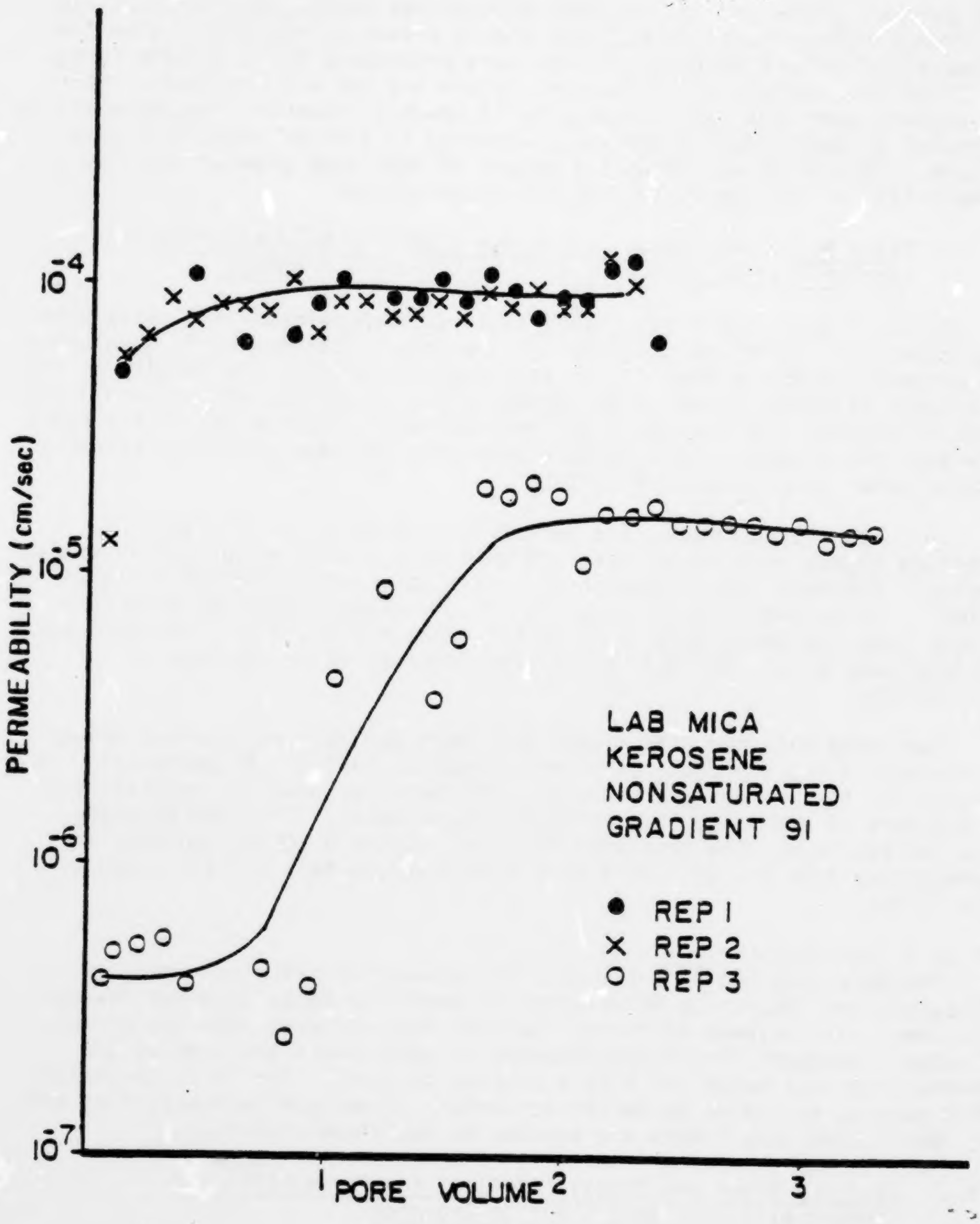
Test data obtained with diesel fuel were variable and did not appear to increase along the patterns of most organics tested. No explanation is offered for the unpredicted behavior. Values were, however, greater by 1 to 2 orders of magnitude than permeability to water. This was also the case for gasoline. The test with motor oil showed a steady increase in permeability from 2×10^{-7} cm/s at 0.5 pore volume to 1.5×10^{-6} cm/s at 3 pore volumes.

4.5.14.1 Discussion--

The data obtained in tests with the commercial petroleum products illustrate the importance of extended permeability tests to allow passage of several pore volumes of fluid. Earlier data indicate that the effects of organic permeant fluids are retarded in tests where the samples are presaturated with water (or 0.01 N calcium sulfate). The tests on unsaturated samples may serve as better estimators of maximum permeabilities that may result when clay liners are exposed to petroleum products.

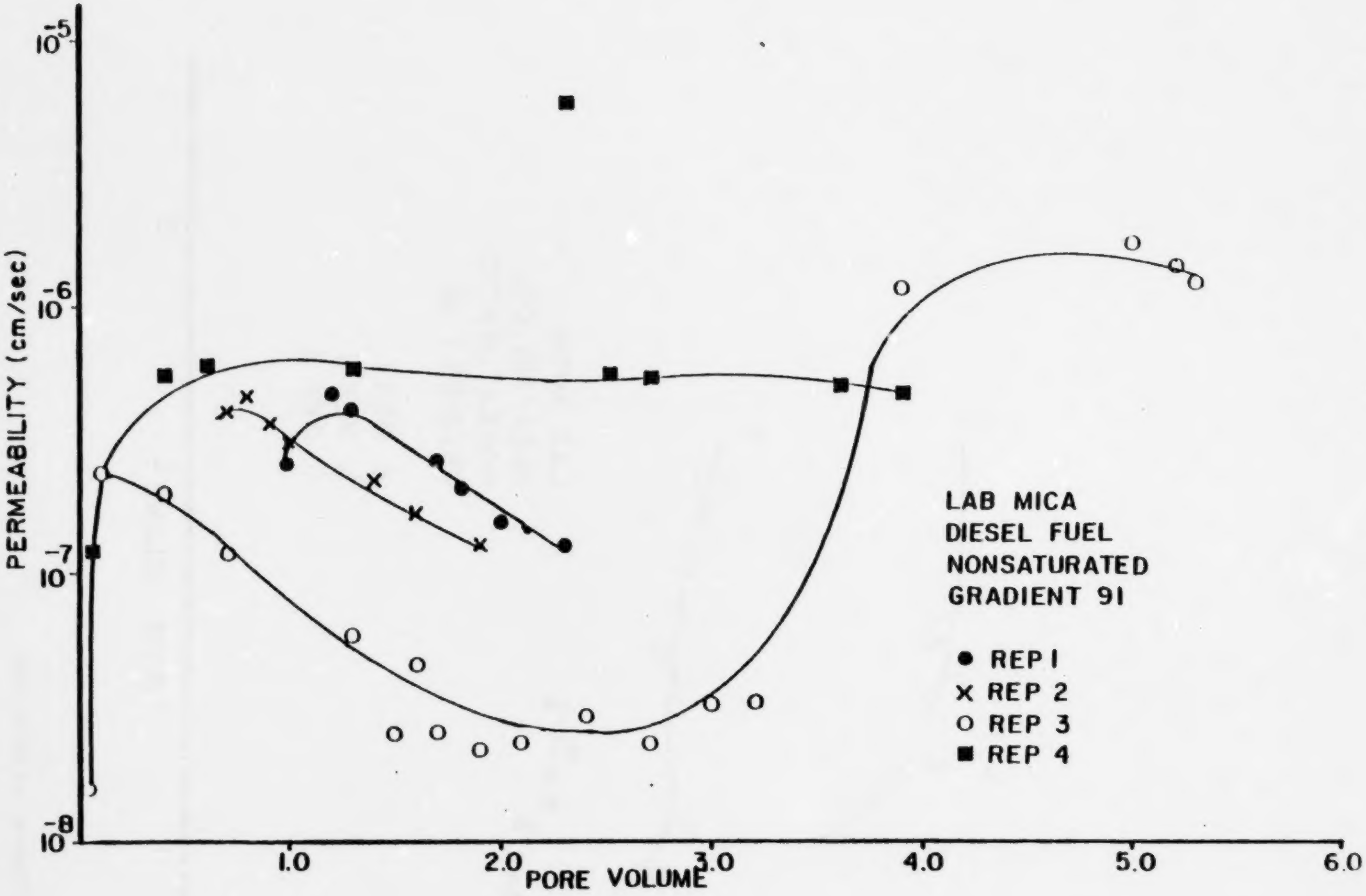
4.5.15 Study by Brown and Thomas (1984) of the Permeability of Commercially Available Clays to Organics

To determine the effect of organic fluids on currently available clays that are sold for sealing and lining impoundments, Brown and Thomas (1984) tested permeabilities of three commercially available clays admixed with sand.



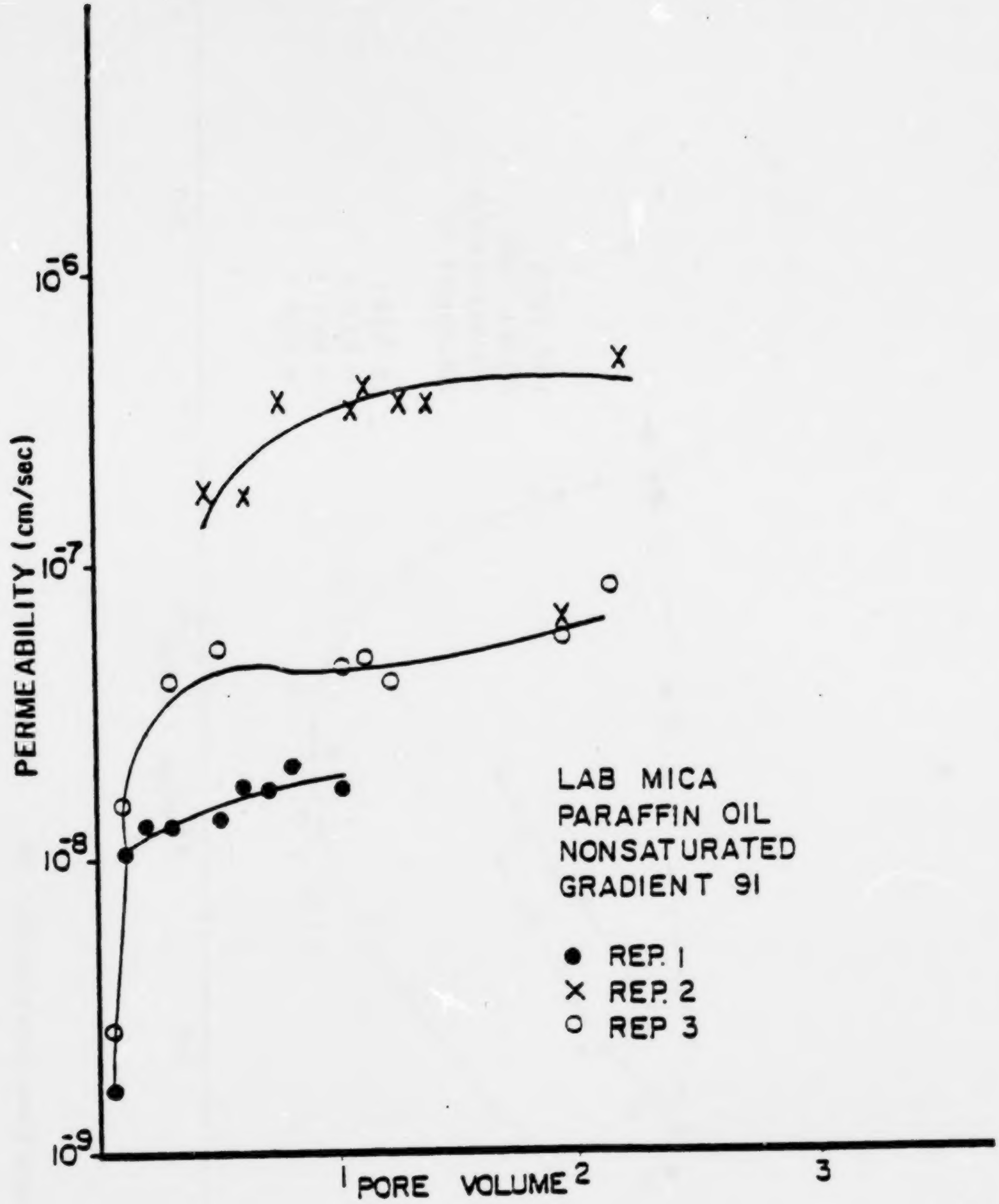
Source: Brown, Thomas, and Green, 1984

Figure 4-16. Hydraulic conductivity versus pore volume for laboratory compacted micaceous soil exposed to kerosene at a hydraulic gradient of 91.



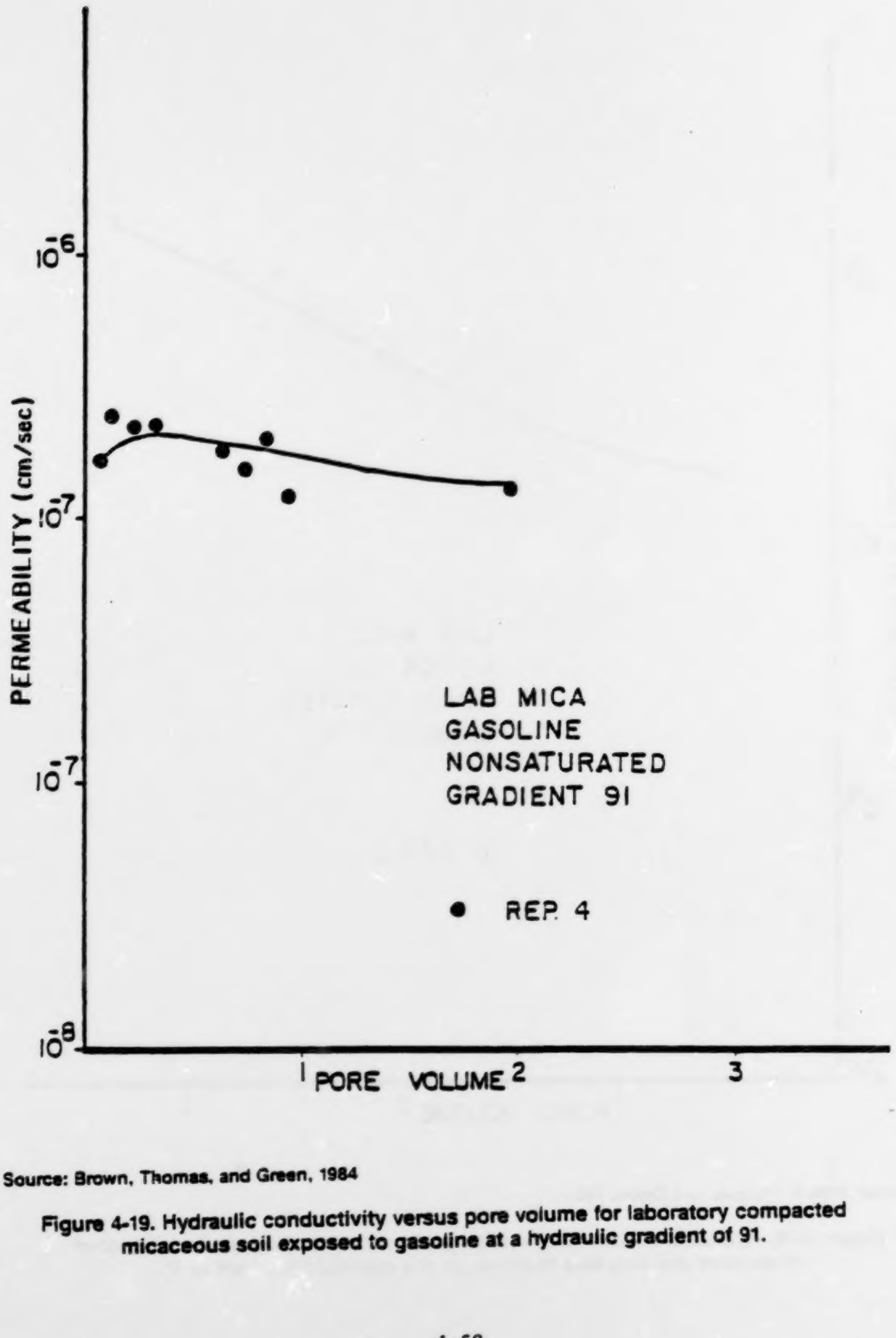
Source: Brown, Thomas, and Green, 1984

Figure 4-17. Hydraulic conductivity versus pore volume for laboratory compacted micaceous soil exposed to diesel fuel at a hydraulic gradient of 91.



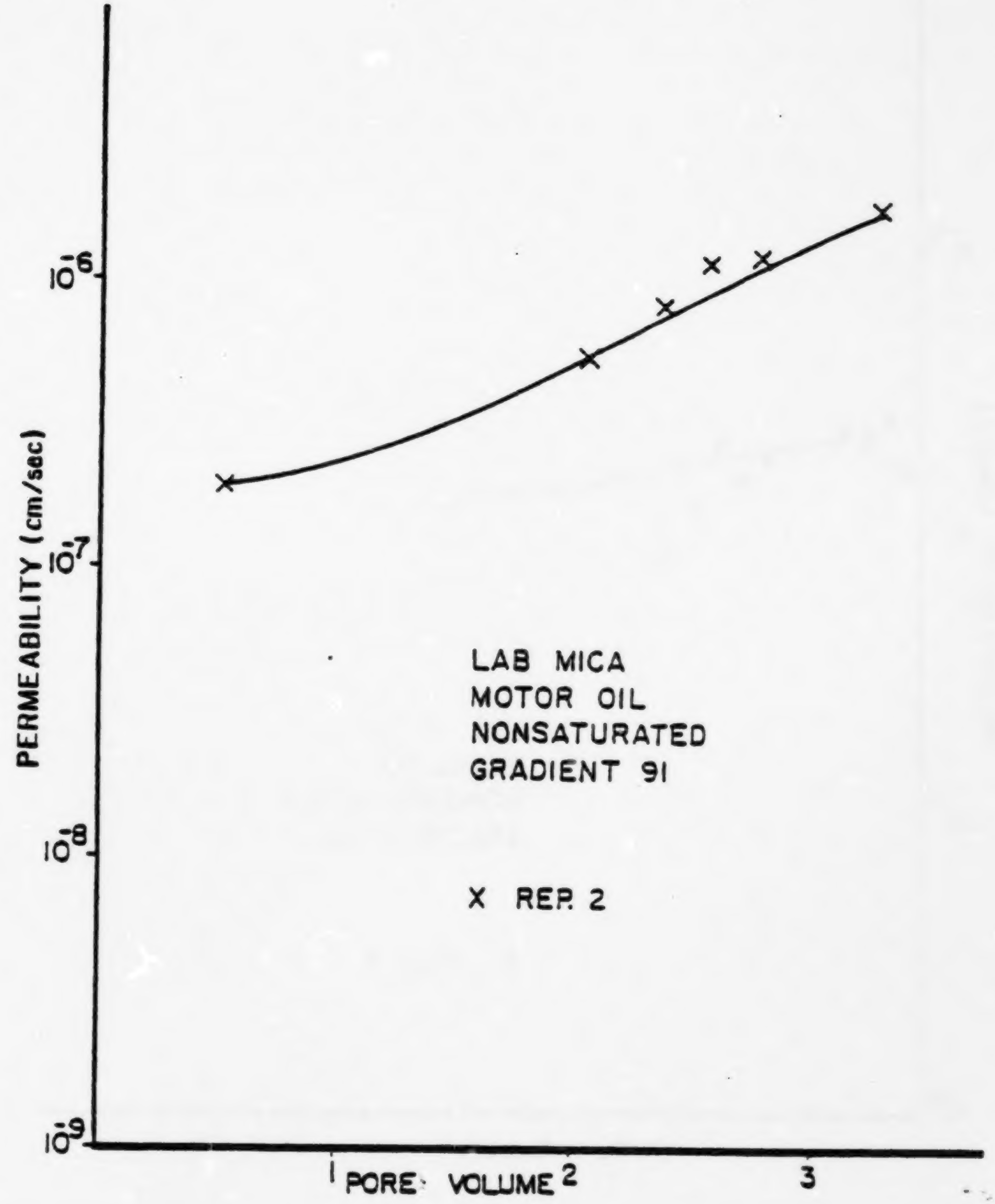
Source: Brown, Thomas, and Green, 1984

Figure 4-18. Hydraulic conductivity versus pore volume for laboratory compacted micaceous soil exposed to paraffin oil at a hydraulic gradient of 91.



Source: Brown, Thomas, and Green, 1984

Figure 4-19. Hydraulic conductivity versus pore volume for laboratory compacted micaceous soil exposed to gasoline at a hydraulic gradient of 91.



Source: Brown, Thomas, and Green, 1984

Figure 4-20. Hydraulic conductivity versus pore volume for laboratory compacted micaceous soil exposed to motor oil at a hydraulic gradient of 91.

4.5.15.1 Test Method--

Each clay was mixed with sand to obtain a permeability to water of about 1 × 10^{-8} cm/s. Smectite is the dominant mineral in soil CC1 (blue bentonite) and soil CC2 (a synthetically treated bentonite); a micaceous mineral is dominant in soil CC3 (Ranger Yellow). The soils were compacted at optimum moisture in 10-cm fixed-wall molds using a mechanical compactor as described in ASTM Procedure 698-70.

The compacted soils were exposed to the test fluid without presaturation. Following a 24-hour equilibrium period, a pressure of 15 psi (equivalent to a hydraulic gradient of 91) was applied to the liquid surface. Leachate passed through the samples was collected at intervals to allow calculation of the soil permeability. Fluids tested in replicate were water (0.01 N calcium sulfate), acetone, xylene, kerosene, diesel fuel, gasoline, and used motor oil. A statistical analysis was performed to establish the variance among the data set for each permeant fluid. Means were separated using a Duncan's Multiple Range test at a significance level of $P = 0.05$.

4.5.15.2 Test Results--

The test results for all permeant fluids are summarized in Table 4-12. All of the organic fluids tested produced dramatic increases over the corresponding permeabilities to water, the increase ranging from 1 to 5 orders of magnitude. Some of the increases, though large, were not statistically significant compared to the permeabilities measured with water due to the large variability seen in the tests. The increases in permeability were significant for all clays permeated with xylene and for both the smectitic clays when permeated by gasoline and kerosene.

Generally the increase in the micaceous clay permeability was less by 1 order of magnitude than the increases seen in the smectitic clays. The untreated bentonite (CC1) showed the greatest permeability increase in response to each organic test fluid. Since the untreated bentonite is the soil that is most subject to shrinkage and the micaceous soil is least subject to shrinkage, these findings are consistent with the theory that shrinkage resulting in greater spacing between peds is responsible for the changes in soil permeability.

4.5.16 Studies Conducted for EPA by Daniel (1983) and Foreman and Daniel (1984) at the University of Texas, Austin

Under a cooperative agreement with EPA, Daniel and others have carried out research to address how the permeability of clay soils is affected by hydraulic gradient and by the test device when the permeant fluid is other than water. Permeability tests were performed using three different types of test devices--flexible-wall cells, fixed-wall compaction mold permeameters, and consolidation cell permeameters--at hydraulic gradients of 10, 50, 100, and 300. The three soils studied were a noncalcareous smectite (Lufkin clay), a mixed-cation illite (from Hoytville, Ohio), and a commercially processed kaolinite (Hydrite R). Fluids tested were water (actually 0.01 N calcium sulfate), methanol, and heptane (Foreman and Daniel, 1984).

TABLE 4-12. MEAN CONDUCTIVITY OF EACH SOIL TO EACH FLUID TESTED
(Brown and Thomas, 1984)

Fluid	CC1	CC2	CC3
Water	3.61×10^{-8} b[a]	2.58×10^{-8} b	1.57×10^{-8} b
Acetone	5.05×10^{-5} b	1.41×10^{-6} b	2.51×10^{-7} b
Xylene	1.76×10^{-4} a	7.28×10^{-4} a	1.00×10^{-4} a
Gasoline	1.96×10^{-4} a	9.07×10^{-5} a	6.19×10^{-5} b
Kerosene	1.49×10^{-4} a	9.10×10^{-5} a	5.68×10^{-5} b
Diesel fuel	5.17×10^{-5} b	4.53×10^{-5} ab	6.29×10^{-7} b
Motor oil	6.13×10^{-6} b	2.13×10^{-6} b	9.48×10^{-7} b

[a] Values in a given column followed by the same letter do not differ significantly ($P = 0.05$).

4.5.16.1 Test Method--

Most of the tests for the project were flexible-wall tests. The fixed-wall and consolidometer tests were performed to allow comparison of the data from the different devices. Soil samples were compacted at or slightly dry of optimum moisture content according to ASTM D-698. Soils were scarified between each lift. Samples to be tested in flexible-wall cells were extruded from the 10.2-cm (4-inch) -diameter compaction mold and trimmed to a height of 9.4 cm (3.7 inches). Similar diameter samples with a height of 11.9 cm (4.7 inches) were tested in the fixed-wall permeameter setup. Samples tested in the consolidation cell were taken from the central portion of a compacted 10.2-cm (4-inch) sample. The sample was trimmed to a height of 1.90 cm (0.75 inch) and a diameter of 6.4 cm (2.5 inches).

Prior to the permeability measurements, the soil samples were saturated with the permeant fluid to be tested. For the flexible-wall tests great care was taken to completely saturate the samples. This was accomplished through soaking under vacuum and backpressuring. During the permeability tests, the backpressure was kept at about 2.8 kg/cm^2 (40 psi) and the cell pressure at 3.87 kg/cm^2 (55 psi). The average effective stress was 1.05 kg/cm^2 (15 psi) for tests at the lower hydraulic gradients; an average effective stress of 1.76 kg/cm^2 (25 psi) was used when the gradient was 300.

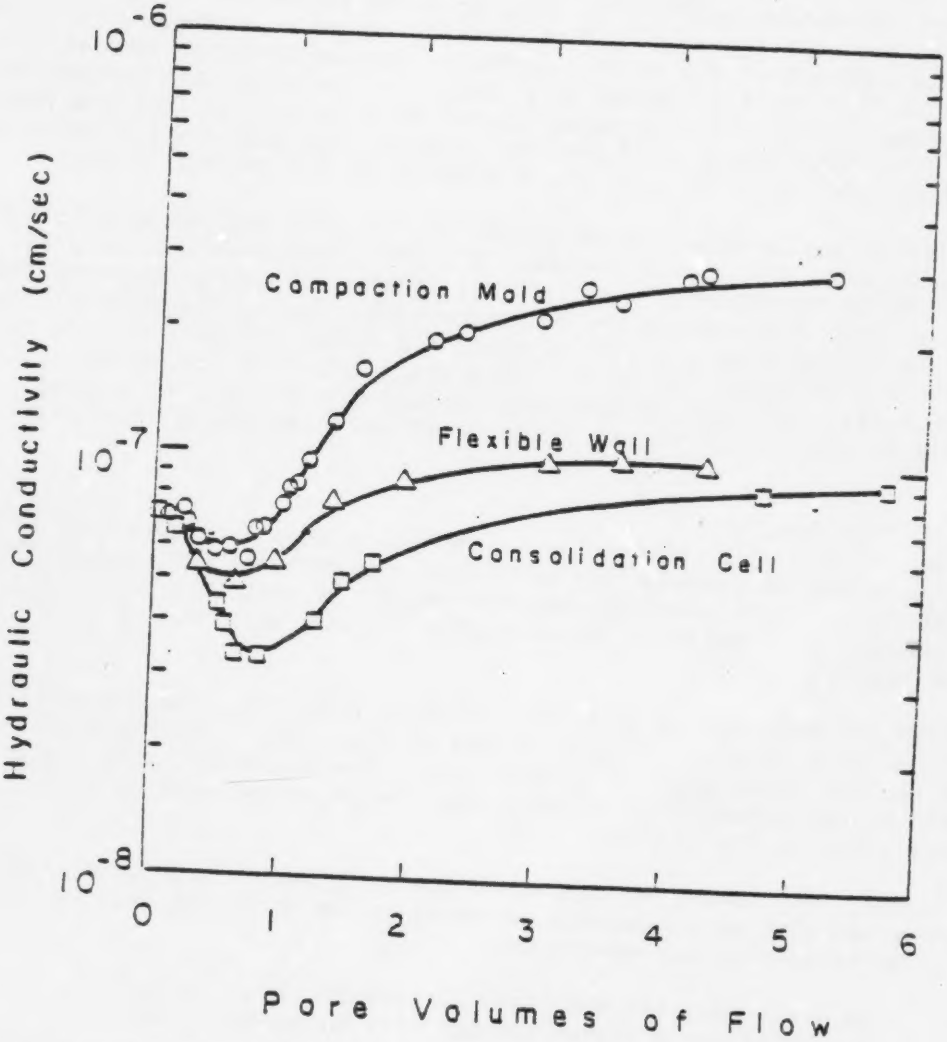
4.5.16.2 Test Results--

Test results with Lufkin clay in the flexible-wall cells showed essentially no change in permeability with time when methanol was used as the permeant fluid. Data were obtained for passage of about 0.9 pore volume. The permeability to methanol was virtually the same as with 0.01 N calcium sulfate (Daniel, 1983).

Methanol test results with the kaolinite show that when permeability is plotted as a function of pore volumes of flow, the curves have similar shapes for the three types of permeameters. The compaction mold yields somewhat higher permeability results than the other two test devices. Test data are shown in Figure 4-21.

Permeability values for the kaolinite at different hydraulic gradients were obtained with water and with methanol in the different test devices. The following conclusions were drawn:

- At high hydraulic gradients (values of 150 or larger), kaolinite has a higher conductivity to methanol than to water regardless of permeameter type.
- At high hydraulic gradients, the flexible-wall and consolidation- cell permeameters yield similar hydraulic conductivities for both methanol and water; kaolinite is roughly twice as permeable to methanol as to water for these two types of permeameters.



Source: Foreman and Daniel, 1984

Figure 4-21. Permeability versus number of pore volumes of flow for kaolinite permeated with methanol at a hydraulic gradient of 250 or 300.

- At high gradients, use of compaction-mold permeameters leads to large scatter in measured hydraulic conductivity. On the average, compaction-mold devices showed kaolinite to be approximately 10 times more permeable to methanol than to water. The lowest values of K measured with the compaction-mold devices are nearly identical to values measured with the other two devices. It is possible that sidewall leakage contributed to the causes . . . .

- With flexible-wall permeameters and two liquids and one soil, hydraulic gradient appears to have little effect on hydraulic conductivity . . . .

- With consolidation-cell permeameters, hydraulic gradient has a very substantial effect on hydraulic conductivity. At a hydraulic gradient of 50, K to methanol is only half of K to water, but at gradients of 200 to 300, K to methanol is twice as large as K to water . . . .

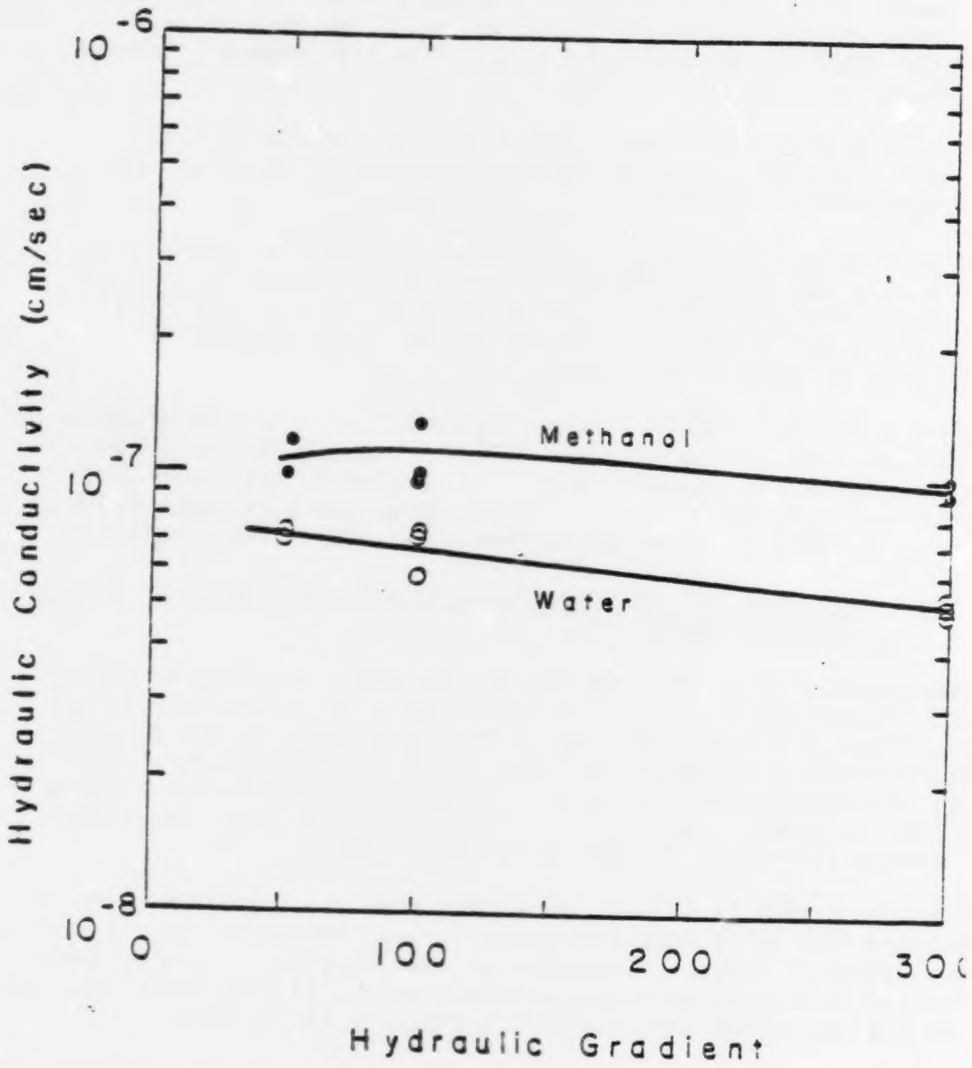
Figures 4-22, 4-23, and 4-24 show the effect of hydraulic gradient on tests with kaolinite in flexible-wall cells, consolidation cells, and fixed-wall permeameters, respectively. The scatter in the fixed-wall permeameter data does not allow conclusions to be drawn although there appears to be a trend to increased permeability with increased gradient.

4.5.17 Tests Conducted for Chemical Manufacturers Association by Daniel and Liljestrand (1984)

In work sponsored by the Chemical Manufacturers Association (CMA), Daniel and Liljestrand at the University of Texas at Austin carried out a laboratory study of the permeability of five clay soils to six aqueous fluids representative of waste leachates or actual liquid wastes. The purpose of the testing was to determine if dilute organic/water mixtures produced the increases in permeability demonstrated by other researchers using concentrated organic liquids as permeant fluids.

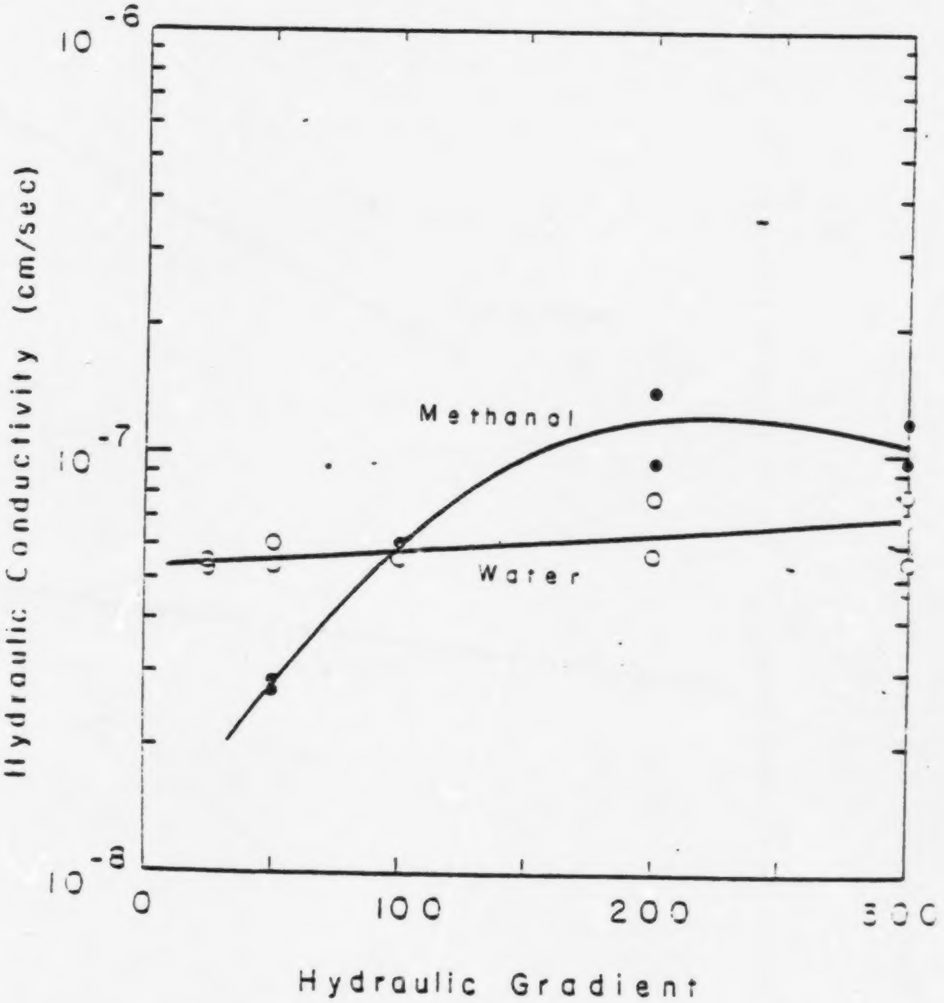
The soils studied included Texas Lufkin clay (also used in tests at Texas A&M) and four soils supplied by CMA (identified as S1, S2, S3, and S4) from actual landfill sites. Properties of the clays are listed in Table 4-13. Permeability tests were not performed on the S1 clay since its index properties and mineralogy were similar to those of the S2 clay.

Permeant liquids used in the tests were: a leachate from a solid-waste landfill (L2); two liquids (L1 and L3) from industrial waste impoundments; an aqueous solution containing methanol at 5 percent (50,000 ppm); an aqueous solution saturated with xylene (near 196 ppm); and the leachate, L1, spiked with chloroform (200 ppm) and trichloroethylene (200 ppm) to simulate a landfill leachate contaminated with chlorinated hydrocarbons. Water used to prepare the dilute methanol and xylene solutions was actually 0.01 N calcium sulfate. Characteristics of the leachates L1, L2, and L3 are listed in Table 4-14.



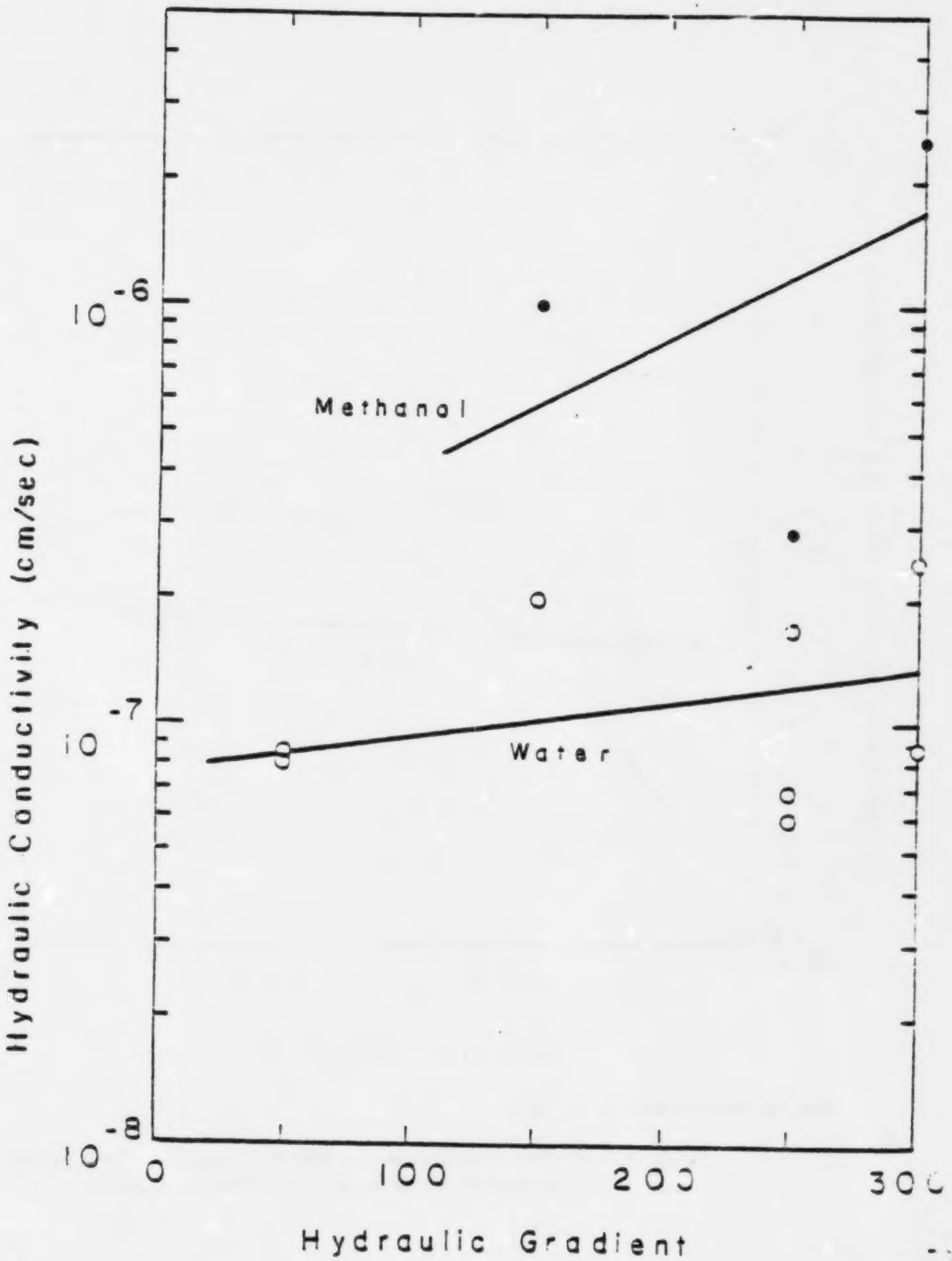
Source: Foreman and Daniel, 1984

Figure 4-22. Permeability versus hydraulic gradient for kaolinite permeated in flexible wall permeameters.



Source: Foreman and Daniel, 1984

Figure 4-23. Permeability versus hydraulic gradient for kaolinite permeated in consolidation cell permeameters.



Source: Foreman and Daniel, 1984

Figure 4-24. Permeability versus hydraulic gradient for kaolinite permeated in compaction mold cell.

TABLE 4-13. PROPERTIES OF CLAY SOILS TESTED BY DANIELS AND LILJESTRAND (1984)

Property	S1	S2	S3	S4	Lufkin clay
Natural water content (%)	23	22	47	32	23
Optimum water content (%)	17	24	31	18	21
Specific gravity	2.73	2.81	2.71	2.71	2.66
Percent finer than #200 sieve	93	93	94	87	81
Percent sand	7	7	6	13	19
Dominant minerals	Illite	Chlorite	Smectite	Quartz	Smectite
Secondary minerals	Chlorite	Smectite, kaolinite	Kaolinite	Illite, kaolinite	Kaolinite, illite
Organic carbon content (% of dry weight)	1.46	0.83	1.39	1.79	0.28
Cation exchange capacity (meq/100 g)	10	20	40	5	25
Plasticity index	21	32	59	16	42

TABLE 4-14. LEACHATE CHARACTERISTICS[a]

Parameter measured or reported	Leachates L1	Leachates L2	Leachates L3
Suspended solids (mg/L)	465	162	107
Dissolved solids (mg/L)	20,202	1,043	4,593
Volatile solids (mg/L)	5,768	121	1,052
Total organic carbon (TOC) (mg/L)	1,440	13	82 120
Chemical oxygen demand (COD) (mg/L)	2,160	12	130
Potential (ORP) (mV)	398	268	496
pH	6.9	7.1	1.5
Conductivity (μmho/cm)	23,700	865	4,620
Metals (mg/L)		b	b
Cr	1.0		
Cu	5.3		
Pb	0.36		
Ni	0.18		
Zn	6.0		
Organics (mg/L)	b	b	
Ethylbenzene			0.120
Toluene			0.035
Nitrotoluene			8
Dinitrotoluene			117
Formaldehyde			14
Dichlorobenzene			8
Aniline			4
Toluenediamine			2

[a] Data from Daniel and Liljestrand, 1984.

[b] Not reported.

4.5.17.1 Test Method--

Permeability tests were carried out as described previously in Section 4.5.16. All testing was performed using flexible-wall permeameters and hydraulic gradients of 150 or 200. Rates of flow were determined from the rate of inflow for each test chamber.

4.5.17.2 Test Results--

The permeability tests were performed for several months, allowing the passage of more than one pore volume of fluid for most soil/leachate combinations. The permeabilities of the soils S2, S3, S4, and Lufkin clay appear to be related to the plasticity index, the permeability decreasing with increasing plasticity. For any one soil, the permeabilities to the various liquids were all about the same. Permeabilities to the leachates did not differ significantly from the permeability to water (actually 0.01 N calcium sulfate). All permeabilities measured were 1×10^{-8} cm/s or lower. In three of the four soils tested, the permeability to 196 ppm xylene was slightly lower than the permeability to the 5 percent methanol. All permeabilities were plotted as a function of pore volumes of fluid passed through the sample.

In addition to the permeability tests, Atterberg limits of the soils were determined using the various test fluids and pure xylene and methanol in place of water. Test fluids L1, L2, L3, and spiked L1 did not significantly alter the plasticity of any of the clay soils during the short-term exposure of the test. Mixing any of the five soils with pure xylene or pure methanol, however, caused a drop in the liquid limit and destruction of the soil's tendency to be plastic. The dilute solutions of methanol and xylene (as used in the permeability tests) had much less significant effects on the plasticity. Only the soil S3 showed a large drop in plasticity when mixed with the 5-percent methanol or the aqueous solution containing 196 ppm xylene.

The significance of Daniel and Liljestrand's findings is that they appear to show that dilute organic/water mixtures are not capable of causing significant changes in the permeability of natural clay liners.

4.5.18 Study by Dunn (1983) of the Effects of Synthetic Lead-Zinc Tailings Leachate on Clay Soils

Dunn (1983) evaluated the effects of a synthetic lead-zinc tailings leachate, of low pH and containing large concentrations of heavy metals, on the permeability of two clay soils. The permeability of the clays was shown to be affected by the waste permeant, with cation exchange and precipitation apparently being the most important processes.

The soils used in the testing, Altamont soil and Rockville soil, had been identified as potential liner construction materials. Altamont soil, a "valley alluvium" with moderate plasticity, is a silty clay soil with 1 or 2 percent shale fragments. Montmorillonite is the predominant clay mineral in Altamont soil. Rockville soil, a yellow-brown silty clay with high plasticity, is composed of the fine fraction from a sand and gravel plant. The predominant clay mineral is kaolin. This soil probably contained microorganisms since some evidence of bacterial growth was noted in soil that was stored for several months.

The permeant fluids tested were tap water, distilled water, and the synthetic lead-zinc tailings leachate. Characteristics of this fluid were:

- Conductivity 1,550 µmho/cm
- pH = 2.6
- Lead = 5.8-15.0 mg/L
- Zinc = 200 mg/L.

The leachate was prepared with zinc sulfate, lead sulfate, and lead nitrate. Sulfuric acid was used to attain the desired pH.

Two tests with each clay were conducted with the synthetic leachate. Permeabilities obtained with the test fluid were compared to the results of samples tested with tap water and with distilled water. All the tests were run at a compacted dry density of about 90 percent of maximum dry density and at a water content approximately 1 to 2 percent above optimum. Tap water was used for the molding water. Soil samples were prepared with static compaction since this method was found to be most appropriate for producing replicate samples at approximately the same dry density and water content. The static compaction involves compressing the soil to a known density with applied hydraulic pressure.

Tests were performed in a triaxial cell at a hydraulic gradient of 50. Samples tested were 3.81 cm (1.5 inches) in diameter and 2.54 cm (1 inch) thick. The samples were presaturated (using a backpressure technique) with the fluid to be tested. Prior to permeability testing, the samples were consolidated at 1.5 kg/cm^2 (21.3 psi) effective stress. Permeability measurements were carried out at an effective stress of 1.25 kg/cm^2 (17.8 psi).

The permeability values measured in both soils exposed to the synthetic leachate were higher than those measured with tap water. Values measured with distilled water were lower than permeabilities measured with tap water. The ultimate effect of the synthetic leachate on the permeability to either soil could not be determined from the tests, however, because the permeability values did not reach steady state during the duration of the test (5,000 minutes or 3.5 days). The tests with tap water did appear to reach steady state during this test time.

For one Altamont sample, the measured permeability to the synthetic leachate was approximately 2 orders of magnitude higher than that of tap water. This appeared to correlate with a long curing period (the period of time from compaction to the beginning of the permeability test). Thixotropic alterations of the soil fabric may have occurred during this interval, resulting in the higher K value.

4.5.19 Studies by Acar and Others (1984) on the Effect of Organics on Kaolinite

In research funded by EPA, Acar, Olivieri, and Field (Acar et al., 1984b and c) and Acar, Hamidon, Field, and Scott (Acar et al., 1984a) studied the effect of four organic fluids on the saturated permeability of Georgia kaolinite. The fluids tested--benzene, acetone, phenol, and nitrobenzene--were chosen because they represent a wide range of dielectric constants. Comparative tests were performed with 0.01 N calcium sulfate. In addition to the pure solvents, 0.1 percent (1,000 ppm) solutions of acetone and phenol prepared in 0.01 N calcium sulfate solutions were tested. This research was carried out at Louisiana State University, Hazardous Waste Research Center.

4.5.19.1 Test Method--

The kaolinite was cured at 32 percent moisture for 1 week before compaction in a Harvard miniature mold at a compactive effort corresponding to standard Proctor compaction. Sample dimensions were restricted to 3.55 cm in diameter and 3.8 to 5.1 cm in height.

Tests were conducted in triaxial cells with continuous backpressure. Hydraulic gradients of less than 100 were used. Backpressures of 414 to 449 kPa (60 to 65 psi) were used to fully saturate the samples prior to the permeability testing. Approximately one pore volume of 0.01 N calcium sulfate was passed through the samples to establish the reference permeability value. The influent liquid was then switched to the organic fluid to be tested. Tests were continued until the permeability readings and the effluent concentrations were stable.

A mercury intrusion method was used to characterize the pore size distribution in the samples before and after permeation with the organic test fluids and with the calcium sulfate (Acar et al., 1984b).

A fixed-wall test with acetone as well as flexible-wall tests at variable effective stresses were also carried out in order to evaluate the test scheme.

Free-swell and liquid limit tests were conducted with the organic solutions and with the standard calcium sulfate to determine if these properties were affected by the chemicals.

4.5.19.2 Test Results--

All tests with chemicals at low concentrations resulted in slight decreases in permeability. It was found, however, that the chemicals diffused through the flexible latex membrane so that the concentration of the organics that actually permeated the soil sample could not be ascertained.

Reference permeabilities in all kaolinite samples tested with pure organics were between 5.0 × 10^{-8} cm/s and 6.0 × 10^{-8} cm/s. When pure organic fluids were introduced into the test cells, an immediate decrease in permeability occurred. For acetone and phenol, this decrease was followed by a permeability increase, the final value stabilizing at approximately double the initial or reference value. In the tests with benzene and nitrobenzene, the permeability decreased until the tests were terminated. Final permeability values measured in these tests were 2 orders of magnitude lower than the reference values.

When acetone was tested in a fixed-wall permeameter, the permeability stabilized at 2×10^{-6} while tests under comparable conditions in flexible-wall cells yielded values between 6×10^{-8} and 9×10^{-8}. The difference in these test results is attributed to sidewall leakage in the fixed-wall test resulting from sample shrinkage during permeation with acetone. The free-swell tests also confirm that shrinkage would occur with acetone permeation.

Both swelling behavior and liquid limit determined with the specific permeant fluid were found to relate to the changes in permeability measured. The liquid limit and free swell were increased significantly with benzene and nitrobenzene, slightly increased with phenol, and decreased with acetone.

Although absolute permeabilities were altered considerably by permeation with the pure organic fluids, the mercury intrusion investigations showed that the size and distribution of pores greater than 80 Å were not significantly affected. Since pores with diameters of less than 80 Å are not expected to contribute appreciably to the total flow, these results suggest that physicochemical properties of the pore fluid close to the clay surfaces lead to variations in flow characteristics.

4.5.20 Finding by Olivieri (1984) of Impermeability of Montmorillonite to Benzene

Olivieri (1984) found that benzene did not penetrate compacted Ca-montmorillonite that was first fully saturated with 0.01 N calcium sulfate solution even at a hydraulic gradient as high as 150. This finding was attributed to hydraulic pressures being less than the required flow initiation pressure to two-phase flow (Acar and Seals, 1984).

4.5.21 Study of Permeability of Clays to Simulated Inorganic Textile Wastes by Tulis (1983)

Tulis (1983) tested Wyoming bentonite, Grolley kaoline, vermiculite, and White Store clay in compaction permeameters with alkaline metal hydroxides. Test solutions were ferrous hydroxide, cupric hydroxide, and manganese hydroxide prepared from 1 N solutions of the respective sulfates by adding sodium hydroxide until a precipitate formed. The final solutions had pH readings as follows: ferrous = 12.9, manganese = 13.0, and cupric = 12.5. Test results show an increase in permeability of the bentonite when it was exposed to the alkaline permeant fluids. The permeability of the kaoline decreased with the application of the bases. Neither the field clay nor the vermiculite showed any significant variation in permeabilities with the test fluid.

Cracks observed in the bentonite indicate that shrinkage was the mechanism responsible for the observed increase in permeability. The decreased permeability of the kaoline was attributed to clogging of the pore space by dispersed clay. All of the leachates from the tests contained silica that had been dissolved by the alkaline solutions.

4.5.22 Tests Conducted by Engineering Consulting Firms for Specific Application (unpublished data)

4.5.22.1 Tests Conducted by the Trinity Engineering Testing Corporation--

The tests discussed below were performed by the Trinity Engineering Testing Corporation at the request of the Corpus Christi, Texas, City Water Department (White, 1976).

4.5.22.1.1 Soil Characteristics and Test Method--A soil sample from a proposed toxic waste landfill site was subjected to permeability testing with isopropyl alcohol, benzene, and charcoal starter fluid. The material tested was a subsurface clay. Composite samples were compacted at optimum moisture content to a height of approximately 91.4 cm (36 inches) at 95.8 percent standard Proctor density in 2.54-cm (1-inch) inside diameter cylinders. Each of the three organic fluids was added to a level of approximately 61 cm (24 inches) above the compacted soil samples. Water was tested in a fourth sample. A constant pressure head of 7.01 m (23 feet) was imposed with compressed air. Liquid level in each test cylinder was recorded every 4 hours until all the liquid had penetrated the full 91.4-cm (36-inch) column of compacted material.

4.5.22.1.2 Test Results--In the experiments with isopropyl alcohol and water as the test fluid, the total drop in liquid level was less than 1 inch over a 100-day period. In the column tested with the charcoal starter fluid, liquid began dripping from the bottom of the test cylinder after 122 days. In the benzene test column, signs of full penetration throughout the clay material were observed after 36 days.

In a second experiment series, two 91.4-cm (36-inch) compacted clay soil samples were tested with benzene. The samples differed only in their moisture content (sample A at 10 percent moisture and B at 20 percent moisture). Sample A showed signs of full penetration of the benzene after 20 hours. Full penetration of benzene occurred after 32 days in sample B, and all liquid passed through the sample in 71 days.

The conclusions drawn from the permeability tests are stated below:

- Under optimum compaction, the clays are highly impermeable to domestic water and, conversely, are very permeable to lighter hydrocarbons.
- When properly compacted to a finished thickness of 91.4 cm (3 feet), the clays will serve as a suitable tank liner for domestic water but will not contain the lighter hydrocarbons.

- Under optimum compaction, a 91.4-cm (3-foot) liner of the clay in a tank 7.01 m (23 feet) deep containing benzene can begin to leak within 36 days.

4.5.22.1.3 Discussion--Although the soil sample used in the Trinity Engineering experiments is not adequately characterized, the results of the test clearly indicate potential for large permeability increases resulting from exposure to concentrated nonpolar hydrocarbons. The apparatus and test procedures used differ substantially from those used in permeability tests conducted by other investigators.

The data also illustrate the importance of moisture content during compaction. The performance of clay-soil liner in contact with chemicals such as benzene could be drastically influenced by the uniformity of the moisture content when the liner material was installed and compacted.

4.5.22.2 Test Data Submitted to Pennsylvania Department of Environmental Resources--

The Pennsylvania Department of Environmental Resources has received data pertinent to clay liner/chemical compatibility. The data pertain to specific sites and specific wastes and were submitted to the State by consulting engineers. Testing procedures vary and information needed to evaluate the data is not always provided in the reports. All test results show the clay liner permeabilities to be "within the range required" after exposure to the wastes tested.

Report A deals with tests to evaluate effects on a liner material in contact with a waste comprised of one part oil contaminated soil and four parts water. For the permeability tests, samples were air dried and then recompacted, at ±2 percent of optimum moisture content, to 95 percent of maximum dry density. Stainless steel molds 10.2 cm (4 inches) in diameter and 11.7 cm (4.6 inches) in height were used. After trimming, the molded sample was transferred to a constant head permeability device. A backpressure was applied to two of the four samples during saturation with 0.01 N calcium sulfate solution. Permeability measurements with the calcium sulfate showed no significant differences between the results for samples with or without backpressure. Thus, it was concluded that the effect of entrapped air was minimal.

After permeability tests with the standard calcium sulfate, the liner samples were placed in contact with the test fluid, sealed, and placed on a rocking table for 30 days. Three of the samples were tested with the waste fluid and one with calcium sulfate solution. After 30 days, the samples were drained and the permeability tests repeated. Reported results are shown in Table 4-15.

No discernible color change was observed after a 30-day exposure to the waste fluid. No shrinkage along the sides of the mold was observed, although a change in height of approximately 0.64 cm (0.25 inch) was reported.

TABLE 4-15. PERMEABILITY TEST RESULTS[a]
(Pennsylvania Case A)

Sample	Coefficient of permeability (cm/s)	
	With 0.01 N $CaSO_4$	After exposure to test fluid[b]
A[c]	1.1×10^{-7}	1.2×10^{-7}[d]
B[c]	5.9×10^{-8}	3.9×10^{-8}
C	9.4×10^{-8}	1.1×10^{-7}
D	1.7×10^{-7}	Not tested

[a] Test data reported to Pennsylvania Department of Environmental Resources.

[b] Test fluid was one part oil-contaminated soil to four parts water.

[c] Samples under backpressure during initial permeability tests.

[d] Control sample--tested after 30-day exposure to 0.01 N $CaSO_4$.

The waste tested--one part oil contaminated soil and four parts water--was not characterized further. Neither the extent of the oil contamination nor the characteristics of the contaminated oil were reported. It may be assumed that either tap water or deionized water was used to prepare the waste fluid. The extent to which the oil or its contaminants would be extracted by the water is unknown but probably very small. Although it is not discussed in the report, the waste material in contact with the clay liner sample may have involved three phases, with oil being the lightest phase.

No indication of the length of the permeability testing procedure or pore volumes displaced is given. The hydraulic heads used in the tests also are not reported. These details may be stated in a letter that is referenced in the report. Soil characterization data were not included.

Report B describes the results of a similar investigation in which liner samples from a disposal site were tested with four different wastes. The permeabilities are reported in Table 4-16. All samples showed slight (twofold) increases in permeability after a 30-day exposure to the wastes. It is notable that "control sets" exposed for 30 days to 0.01 N $CaSO_4$ showed similar increases.

Soil properties are listed below:

Cation exchange capacity: 11.1 meq/100 g soil
Predominant exchangeable cation: Calcium
Major mineral fraction: Alpha quartz
Other minerals (trace to minor): Microcline, Adularia, Muscovite, Kaolinite
Percent clay size (0.002 mm): 13
Percent silt size (0.002 mm -0.05 mm): 30
Percent sand size (0.05 mm -2.0 mm): 22
Percent larger than 2.0 mm: 35
Liquid limit (percent water): 31
Plastic limit (percent water): 22

Wastes used in the tests were not characterized beyond the description given in the table. Presumably, they were diluted with water as was done in Case A, but this is not stated in the report. The duration of the tests, pore volumes replaced, and hydraulic head are not provided in the report.

Report C provides permeability data on a soil sample tested with chrome ore leachate. The leachate (pH = 13.0) contained 1,400 mg/L total chromium and 1,200 mg/L hexavalent chromium. A constant head test, conducted at a pressure of 20 psi, gave a permeability of 1.2×10^{-8} cm/s. With the falling head method, a permeability of 2.2×10^{-8} cm/s was measured.

Soil characterization and details of the test method were not provided in the brief report.

TABLE 4-16. PERMEABILITY TEST RESULTS[a]
(Pennsylvania Case B)

Sample	Coefficient of permeability (cm/s)		Waste
	Before exposure to waste	After exposure to waste	
A	1.4×10^{-8}	2.0×10^{-8}	Electric furnace baghouse dust
B	1.4×10^{-8}	2.1×10^{-8}	Tar decanter sludge (high in organics)
C	1.8×10^{-8}	3.0×10^{-8}	Neutralized pickle liquor rinse water sludge
D	1.8×10^{-8}	3.0×10^{-8}	Hot strip mill recycle system sludge (high in oil)

[a] Test data reported to Pennsylvania Department of Environmental Resources.

4.5.22.3 Studies Sponsored by Waste Management, Inc.--

Waste Management, Inc., has sponsored a series of studies to determine compatibility between landfill leachate and in situ clay soil, which functions as the landfill liner. (Weaver and Brissette, Canonie Environmental Services Corporation, 1982). Permeability data were obtained in triaxial devices. Each report includes details of the sampling effort, soil characterization data, and the permeability test results. One important conclusion by Canonie is as follows:

> . . . It has been our experience that in concentrations of less than 1 percent organics have no significant impact on soil permeability.

The laboratory permeability results presented in each of the reports indicate that no appreciable change in permeability will occur on soils at the specific sites due to contact with the waste leachate obtained from the facility.

4.5.22.4 Test Data from D'Appolonia Consulting Engineers, Inc.--

D'Appolonia Consulting Engineers, Inc. (D'Appolonia), has performed numerous permeability studies involving site-specific soil samples and leachates. Projects have involved slurry trench walls as well as liners for landfills. In 1983 D'Appolonia compiled their permeability test data from 14 projects (D'Appolonia Consulting Engineers, Inc., 1983). Leachates tested in the D'Appolonia projects were frequently high in salts, and some were highly acidic. All data were obtained from triaxial tests. Permeability data from the various projects are summarized below.

Project A--No significant changes were observed in long-term permeability tests on soil samples containing 2 percent or 3 percent commercial bentonite when they were subjected to an aqueous waste fluid of pH 7. The fluid, collected during dredging operations and shipped to D'Appolonia by the client, contained minor concentrations of a number of inorganic salts (sulfate, chloride, fluoride). Specific conductance was reported as 1,710 µmho/cm.

The permeability tests involved the exchange of up to 4.7 pore volumes with test times of 24 to 76 days. The hydraulic gradients used varied from 36 to 168; cell pressures were 1.0 to 1.5 kg/cm^2.

Project B--Extensive permeability tests were performed on reddish-brown clay samples (undisturbed Shelby tube samples) using a stabilized pH 4 fluid. The fluid was prepared in the laboratory using pond water and waste from the site. The most significant characteristic of the stabilized pH 4 fluid was the specific conductance at 94,000 µmho/cm at 25 °C; pH was 3.88. It should be noted that Shelby tube samples may not always be compacted to exactly the same density so that slight deviations in measured permeability are not considered to be significant.

Although all permeabilities measured were below 1×10^{-7} cm/s, a trend toward increased permeability was apparent as more pore volumes were exchanged. Changes in soil chracterization were also in evidence as more pore volumes of test permeant fluid were passed through the samples. Elevated hydraulic gradients 110 and 450 were used for the tests, and cell pressures were 2.5 and 4.5 kg/cm^2. Test times ranged from 100 to 350 days. In three of the samples, more than 12 pore volumes were exchanged.

Project C--A waste fluid characterized by high salt concentration and high total organic carbon was tested in four soil samples. The samples contained varying proportions of two soils--a fine-to-coarse sand, and a sandy clay with 1 to 3 percent commercial bentonite.

Permeability tests were run at a hydraulic gradient of 20. After initial permeability increases in some samples, the permeabilities decreased. Samples were tested for 692 days. For each sample, between 2 and 13 pore volumes were exchanged. The permeability decrease noted was approximately 1 order of magnitude for a cement bentonite sample. For the other three soil mixtures, permeability decreased by a factor of 2 to 4.

The constituents present in the permeant are not known precisely. Available chemical analysis data indicate that the chemical concentrations in the waste material vary considerably from year to year.

Project D--Slight decreases in permeability were reported for clay-soil samples (with bentonite) exposed to contaminated groundwater samples taken from piezometers. Soil samples were 25 percent flyash, 25 percent uncontaminated clay, 25 percent contaminated clay, and 25 percent silt. Bentonite (1 percent) was added.

Two permeant fluids were tested; pH values were 10.57 (Sample A) and 8.8 (Sample B). Specific conductance was reported at 35,300 and 13,200 µmho/cm at 25 °C.

Samples were tested at a gradient of 190 with cell pressure at 1.5 kg/cm^2. Test duration was 90 days with more than three pore volumes exchanged. Final permeabilities were determined to be below 1×10^{-8} cm/s.

Project E--Permeability tests were run on a silty clay soil from a proposed facility with four waste leachates as the permeant fluid. Soil samples were compacted in the laboratory (95 percent of standard Proctor density). Chemical characteristics of the waste fluids tested are shown in Table 4-17.

Permeability tests were performed at a hydraulic gradient of 47 and cell pressure of 0.75 kg/cm^2. Total test time was 40 to 50 days. The results of the permeability tests are given in Table 4-18. All samples were saturated with 0.01 N calcium sulfate solution prior to introduction of the waste fluid.

Project F--Permeability data on undisturbed Shelby tube samples tested with tap water showed permeability values ranging from 6.1×10^{-8} to 2.0×10^{-9} cm/s with an average of 1.0×10^{-8} cm/s. Fifty-seven samples were tested with total test time varying from 6 to 10 days. Hydraulic gradient was 25 to 50, and cell pressure was 3.5 to 5.5 kg/cm^2.

TABLE 4-17. CHEMICAL CHARACTERISTICS OF WASTE PERMEANTS, PROJECT E[a]

		Waste leachate permeant fluid[b]			
Parameter	Units	No. 3	No. 4	Sludge	Composite leachate
Column test designation		P-1	P-2	P-3	P-4
Ph	pH units	12.63	11.23	8.37	11.23
Specific conductance	µmho/cm at 25° C	20,500	11,900	10,100	11,000
Filterable residue at 180 °C	mg/L	12,100	8,620	8,720	8,380
Acidity	mg/L $CaCO_3$	0	0	0	0
Alkalinity	mg/L $CaCO_3$	4,140	130	20	224
Phenolpthalein alkalinity	mg/L $CaCO_3$	2,660	90	2	143
Chloride	mg/L	1,600	2,300	570	1,800
Sulfate	mg/L	50	2,550	500	2,250
Dissolved metals:					
Cadmium	mg/L	0.02	<0.01	<0.01	<0.01
Calcium	mg/L	610	170	1,800	505
Chromium (hexavalent)	mg/L	450	14	0.06	21
Chromium (total)	mg/L	880	16	0.08	52
Iron	mg/L	<0.1	<0.1	<0.1	<0.1
Lead	mg/L	0.49	0.32	<0.01	0.01
Magnesium	mg/L	<0.1	0.4	135	2.8
Manganese	mg/L	0.07	0.04	0.14	0.06
Nickel	mg/L	0.23	0.10	0.46	0.10
Selenium	mg/L	1.86	0.196	<0.001	0.342
Sodium	mg/L	945	1,860	12	1,350
Zinc	mg/L	3.82	0.12	0.02	0.03

[a] Data from D'Appolonia Consulting Engineers, 1983.

[b] Leachates were generated from various wastes by 1:4 shake extraction of solid waste with water.

[c] Composite leachate obtained by mixing four waste leachates in the following proportion:

No. 3 leachate - 5 percent
No. 4 leachate - 75 percent
No. 5 leachate - 5 percent (composition not specified)
Sludge leachate - 15 percent.

TABLE 4-18. RESULTS OF PERMEABILITY TESTS, PROJECT E[a]

Sample fluid	Permeant	Pore volumes exchanged	Permeability (cm/s)
P-1	0.01 N $CaSO_4$	-	1.6×10^{-7}
	No. 3 leachate	0 - 3.2	1.7×10^{-7}
		3.2 - 12.7	1.4×10^{-7}
P-2	0.01 N $CaSO_4$	-	1.1×10^{-7}
	No. 4 leachate	0 - 8.8	1.3×10^{-7}
P-3	0.01 N $CaSO_4$	-	1.4×10^{-7}
	Sludge leachate	0 - 4.7	1.5×10^{-7}
		4.7 - 13.5	2.4×10^{-7}
		13.5 - 17.2	4.2×10^{-7}
P-4	0.01 N $CaSO_4$	-	1.1×10^{-7}
	Composite leachate	0 - 0.8	1.2×10^{-7}
		0.8 - 5.6	8.1×10^{-7}

[a] Data from D'Appolonia Consulting Engineers, Inc., 1983.

Project G--Slurry wall test data were provided for two different leachates identified as low pH or high pH. A commercial bentonite product was added to a clay-sand mix. Sixty-day tests at hydraulic gradients of 170 to 205 and cell pressure at 1.5 kg/cm^2 gave permeability values below 7 $\times 10^{-8}$ cm/s. Between 19 and 47 pore volumes were exchanged in the tests.

Project H--Potential clay liner materials were tested with a permeant fluid generated in the laboratory by leaching a young slag from a pilot plant. The pH of the leachate was 4.90; specific conductance was 2.70 µmho/cm at 25 °C.

Hydraulic gradients of 25 and 100 and cell pressures of 0.75 and 1.5 kg/cm^2 were used in the permeability tests. Distilled water was used to saturate the samples. Small permeability decreases were observed in tests with the slag leachate. Total testing time was approximately 2 to 3 months.

Project I--Groundwater spiked with several chlorinated hydrocarbons was used as the permeant fluid in tests on undisturbed Shelby tubes samples. The total concentration of chlorinated ethanes was 500 ppm. The soil samples tested were comprised of smectite (50 to 75 percent), kaolinite (10 to 25 percent), vermiculite (10 to 25 percent), mica (10 to 25 percent), and quartz (10 to 25 percent).

Permeability tests were conducted at a hydraulic gradient of 150 and a cell pressure of 1.5 kg/cm^2. Four samples were tested for up to 140 days with a maximum of 24 pore volumes exchanged. Distilled water was used as the initial permeant fluid. Slight decreases in permeability were observed with the spiked groundwater as permeant fluid.

Project J--Two soil samples were tested with a highly acidic waste fluid (pH = 1.5) that was collected from waste ponds. Other significant characteristics of the fluid were specific conductance = 22,200 µmho/cm at 25 °C and sulfate = 15,000 ppm.

Slight decreases in permeability were observed after exchange of more than 10 pore volumes. Hydraulic gradients of 20 to 100 were used with cell pressures of 0.7 to 2.0 kg/cm^2.

Project K--Three waste fluids were tested with a sandy soil mixed with a commerical bentonite product. Significant characteristics of the waste fluids are shown below.

Parameter	Waste fluids		
pH	8	9	10
°K	6.65	7.65	5.00
Specific conductance (µmho/cm at 25 °C)	920	1,000	430
Sulfate (ppm)	470	460	180

No significant change in permeability occurred when the pH 9 fluid was tested with the sandy clay soil. The test involved exchange of 7.4 pore volumes. A slight increase in permeability was noted with the pH 8 fluid tested with the sandy clay soil after 4.7 pore volumes were exchanged. Waste fluid at pH 10 tested with the sandy clay soil mixed with bentonite decreased permeability after exchange of 4.2 pore volumes.

A hydraulic gradient of 22 was used for the tests involving the waste fluids and soil. A hydraulic gradient of 100 was imposed on the soil bentonite tested with pH 10 fluid. Cell pressure was 1.1 kg/cm^2. The pH 8 and 9 samples were tested at a hydraulic gradient of 22 and a cell pressure of 1.0 kg/cm^2.

Project L--Two waste fluids with neutral pH were used in tests with soil mixed with approximately 1 percent treated bentonite from three vendors. The only notable characteristic of the waste fluids was specific conductance. The permeability data are summarized in Table 4-19. The hydraulic gradient used in the tests was 80 to 90; cell pressure was 1.0 kg/cm^2.

Project M--Permeability tests were performed on 20 soil samples to determine the effect of a permeant fluid of pH 1.5. The fluid, collected from waste ponds at a disposal site, had a high salt concentration (specific conductance = 22,200 µmho cm at 25 °C; sulfate = 15,000 ppm).

The only permeability increases noted were for a soil characterized as silt stone and one characterized as sand stone. For these samples, permeability increases were just less than 1 order of magnitude after passage of approximately 12 pore volumes. Hydraulic gradients used in the tests were 15 to 285; cell pressures were 0.75 to 4.00 kg/cm^2.

Project N--Permeability studies on three composite soil samples were conducted. Three waste leachates used as the permeant fluid were prepared in the laboratory by extracting tailings from a pilot plant. Fluids tested were characterized by pH (3, 6, or 9).

Initial permeabilities were determined with groundwater from the proposed site. The results of the permeability tests are shown in Table 4-20. Permeability increases of approximately 1 order of magnitude were observed in the glacial till samples tested with permeant fluids of pH 6 and 9.

4.5.23 Tests Reported by Bentonite Companies

4.5.23.1 American Colloid Company--

The American Colloid Company produces Volclay soil sealants. These products are a special type of high swelling sodium montmorillonite that has been treated by a proprietary process to render the material unreactive toward most chemical materials. American Colloid Saline Seal 100 is a patented product intended for use in containing wastes with high levels of dissolved salts, acids, or alkali.

TABLE 4-19. RESULTS OF PERMEABILITY TESTS, PROJECT L[a]

	Waste fluid A (1,900 µmho/cm at 25 °C		Waste fluid B (3,800 µmho/cm at 25 °C)	
	$K_{max}/K_{initial}$	Pore volumes replaced	$K_{max}/K_{initial}$	Pore volumes replaced
Cement bentonite	1.0	6	1.0	13
Aqua gel	2.2	8	6.9	7
Saline seal	1.6	7	3.9	13

[a] Data from D'Appolonia Consulting Engineers, Inc., 1983.

TABLE 4-20. INITIAL AND FINAL PERMEABILITIES DETERMINED IN TRIAXIAL CELL TESTS WITH LEACHATES, PROJECT N[a]

Sediment sample	Leachate permeant pH	Laboratory permeability @ 20 °[b] (cm/s)	
		Initial with site groundwater	Final with waste leachates
Glacial till (Composite No. 1)	3	5.5×10^{-8}	1.4×10^{-7}
	6	3.8×10^{-8}	3.4×10^{-7}
	9	5.7×10^{-8}	5.6×10^{-7}
	9	3.6×10^{-7}	5.7×10^{-7}
Stratified drife (Composite No. 2)	3	1.8×10^{-5}	1.3×10^{-5}
	6	1.5×10^{-5}	1.2×10^{-5}
	9	1.3×10^{-5}	1.3×10^{-5}
	9	2.1×10^{-5}	2.1×10^{-5}
4% Bentonite/till	3	1.0×10^{-10}	1.5×10^{10}[c]
Admixture	9	1.0×10^{-10}	1.5×10^{-10}[d]

[a] Data from D'Appolonia Consulting Engineers, Inc., 1983.

[b] Permeability calculations based on final column sample dimensions.

[c] Determined as 83.1 percent saturation based upon final moisture content measurements.

[d] Determined as 83.4 percent saturation based upon final moisture content measurements.

American Colloid has conducted permeability tests of Saline Seal 100 with gasoline, kerosene, and 1,1,2-trichloroethane. The trichloroethane tested was waste solvent that had been contaminated with other uncharacterized materials. It was said to be predominantly trichloroethane (Jepsen, 1983).

The American Colloid tests were conducted with a fixed-wall compaction permeameter. Inside walls of the cylinder were coated with a thin film of slurry to provide a barrier against capillary effects along the wall. Samples were compacted to at least 90 percent Proctor. Soil samples were either 5.1 or 10.2 cm (2- or 4-inch) -thick cores and consisted of a uniform standard silica sand mixed with between 6 and 15 percent bentonite (dry weight). Test samples were prehydrated with deionized water for at least 48 hours, and provision was made for deairing. After the test fluid was added to the permeameter, head loss was recorded periodically until stable readings were established. The results of these permeability tests are shown in Table 4-21.

4.5.23.2 Federal Bentonite--

Another major bentonite company, Federal Bentonite, produces, among other bentonite products, petroleum tank farm sealants. The products are made by treating sodium-bentonite with specific polymers in order to obtain the desired sealing characteristics. PPS-21 is a free-flowing granular bentonite product designed to promote an impermeable barrier in the event of failure or leak in petroleum tank farms.

Permeability tests with water and with kerosene were performed on 5.1 cm (2-inch) samples of test soil consisting of PPS-21 mixed with washed beach sand. Samples were prehydrated with deionized water under a 136-cm head prior to introduction of the kerosene. Tests results are summarized in Table 4-22.

4.5.23.3 Discussion--

The behavior of polymer-treated bentonites over a long time period is not demonstrated in the test results presented here. Although the duration of the American Colloid tests exceeded 40 days, only a fraction of a pore volume of fluid was displaced during the tests. The number of pore volumes was not expressed in the data presented by Federal Bentonite.

Suggestions that the polymers in the treated bentonites will degrade after 3 to 4 years have been made, and at least two laboratories claim to have data in support of this time-degradation behavior (Beattie, 1983; Zlamal, 1983). Many applications for the treated bentonites involve short-term performance requirements. In long-term applications, such as barriers for landfills, the time-degradation issue could have serious implications. The long-term viability of treated bentonite seals needs to be verified.

TABLE 4-21. EFFECT OF CONCENTRATED ORGANICS ON A TREATED BENTONITE SEAL[a]

Organic permeant	Test duration (days)[b]	Pore volumes displaced	Permeability (cm/s)
Gasoline	68	0.6	4.7×10^{-7}
Kerosene	40.2	0.16	4.7×10^{-8}
1,1,2-Trichloroethane (waste)	71	0.58	4.2×10^{-7}[c]

[a] Unpublished data on Saline Seal 100 from American Colloid Company, personal communication, January 23, 1983.

[b] All tests conducted using a hydraulic head of 76.2 cm (2.5 feet).

[c] Permeability of prehydrated soil was 1.5×10^{-7} cm/s prior to addition of organic permeant.

TABLE 4-22. PERMEABILITY (cm/s) OF A TREATED BENTONITE SEAL TO KEROSENE[a,b]

Sample	Prior to addition of kerosene	After exposure to kerosene for 7 days	After exposure to kerosene for 42 days
Sample 1: (Prehydrated for 24 h)	5.1×10^{-8}	3.4×10^{-8}	2.5×10^{-8}
Sample 2: (Prehydrated for 48 h)	3.2×10^{-8}	2.2×20^{-8}	1.5×10^{-8}
Sample 3: (Prehydrated for 72 h)	2.0×10^{-8}	1.3×10^{-8}	1.6×10^{-8}
Sample 4: (Prehydrated for 96 h)	1.3×10^{-8}	1.1×10^{-8}	9.6×10^{-9}

[a] Data from Federal Bentonite (1983) on tank farm sealant PPS-21.

[b] Tests conducted under a standard 136-cm head using a falling head permeameter.

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CHAPTER 5

CURRENT PRACTICES: CLAY LINER DESIGN AND INSTALLATION

This chapter describes the state-of-the-art knowledge and techniques currently used for clay liner design and installation. Information in this chapter was compiled from a variety of sources, including a review of existing literature on design and installation for various applications and interviews with key personnel from selected engineering firms, waste management companies, government agencies, professional associations, and contractors, all of whom are experienced in various aspects of clay liner installation and construction. The number of clay-lined facilities designed specifically for hazardous waste containment and information on these sites are somewhat limited. In addition, the design and installation practices for clay liners for other applications are similar to those for hazardous waste facilities (though possibly not as rigorous in design and installation procedures). For these reasons, information from these applications is included as well.

Clay liner construction utilizes equipment and techniques similar to those used in other earthwork construction projects. However, the soil compaction practices used to achieve low permeability employed in clay liner construction differ from compaction practices used for strength and stability, as used in foundations and roadbeds. For cohesive soils (i.e., clays), maximum strength is achieved when soil is compacted at dry of optimum moisture content (see Chapter 3 for a discussion of compaction parameters), but the lowest permeability is achieved 2 to 3 percent wet of optimum moisture content. Therefore, when clay liners are compacted, it is necessary to specify and control carefully compactive effort, density, and moisture content to ensure that the desired permeability is achieved.

The first two sections of this chapter describe current practices for designing and constructing clay liners. Literature reviews and interviews with design engineers and contractors indicate that construction quality assurance (CQA) and construction quality control (CQC) are considered among the most important elements of successful facility construction, with clay liner failures often attributed to deficiencies in these areas. Because of the importance of CQA and CQC activities during the construction of clay liners, this subject is covered separately in the third section of this chapter, including a description of the element of a CQA plan. Common problems encountered during clay liner construction and solutions to these problems are tabulated in the final section of this chapter.

5.1 DESIGN

The fundamental aspects that must be considered during the design of a clay liner are:

- Stability of the liner against major earth movements such as slope failure, settlement, and bottom heave
- Resistance of the liner to fluid flow (i.e., permeability)
- Compatibility of the liner material with the wastes it is meant to contain
- Long-term durability of the liner.

The design effort applies standard geotechnical engineering practices to address these considerations and thus to meet the special performance requirements of clay liners.

Clay liner design is very site and material specific. Waste volume requirements and waste characteristics must be considered during the design effort, and facility designs must be tailored to the site-specific conditions. The design effort may be divided into the following activities:

- Site investigation
- Liner material selection and characterization
- Facility design
- Preparation of construction specifications and the quality assurance (QA) plan.

Site investigation, liner material selection, and liner material characterization are done prior to facility design. Preparation of construction specifications and the QA plan follow facility design. Most of the design engineers interviewed during the course of this project emphasized that design activities usually continue through the construction of the clay liner because unexpected situations often arise that necessitate modifications in the original plans.

5.1.1 Site Investigation

As with any earthwork project, a clay-lined hazardous waste containment facility must be designed to be compatible with the geological conditions at the specific site. For this reason, the first step in the design effort is a comprehensive evaluation of the site. Ideally, sites should be selected according to the suitability of the in situ earth materials for containing hazardous wastes. However, in actual practice this is not always possible. Constraints such as land use, zoning laws, land ownership, and distance to waste generators often result in sites being selected for reasons other than their technical suitability for containing wastes. This fact, combined with the subsurface heterogeneity and spatial variability that is the rule in most geologic environments, makes adequate site investigation a critical part of facility design.

5.1.1.1 Purpose--

Site investigations are conducted to delineate a site's topography, subsurface geology, and hydrogeology. Topography influences facility configuration and drainage system design (runon/runoff control). Subsurface site investigations are necessary to determine whether soils suitable for liner material are available at the facility site or whether it is necessary to identify and investigate borrow sources. In addition, knowledge of in situ soil properties is important for foundation design. Soil characteristics influence selection of the method of slope stability analysis appropriate for facility design, are a necessary input to stability analyses, and determine the necessity for special design measures to control settlement or to ensure maximum protection against contaminant migration.

Hydrogeologic information about the site is important for the proper monitoring of well placement. From a design standpoint, it is important to determine the depth to water table for the site, including seasonal variability. Some States require a specific thickness of unsaturated soil between the facility base and the water table. For sites with high water tables, this can necessitate aboveground facility design. In States where facilities can extend below the water table, high groundwater levels can necessitate special intergradient (below water table) designs. Groundwater elevations are necessary for assessing liquefaction potential for in situ soils where significant seismic ground motion can occur. Hydrogeologic investigations also are necessary to locate, identify, and delineate hydrologic pathways (e.g., fractures and sand seams) at the site so that provisions for sealing them can be incorporated into the facility design. These pathways can contribute to rapid migration of wastes from the facility if a liner leak occurs. In addition, when liners constructed below the groundwater table intersect these pathways, hydraulic pressures can build against the outside of the liner. In unfilled facilities this can result in heaving, slope failure, and liner rupture.

5.1.1.2 Approach and Methodology--

The investigation of a facility site should address the following:

- Regional and site-specific investigations to relate the site geology to the regional geological picture.

- Topography, including drainage patterns.

- Analyses of representative soil samples. Important tests can include Atterberg limits, particle size distribution, shrink/swell potential, cation exchange capacity, total organic carbon, mineralogy, shear strength, dispersivity, compressibility, consolidation properties, density and moisture content, Proctor density, laboratory (compacted) permeability, and chemical compatibility.

- In-place soil characteristics including depth to bedrock, in-place permeability, and the presence of features that can act as failure planes or hydrologic pathways (e.g., slickensides, fractures, faults, silt and sand lenses and seams, and root holes).
- Bedrock characteristics including type, form, fractures, solution cavities, and joints.
- Hydrogeologic site characteristics including depth of the water table, horizontal and vertical flow components, hydrogeologic pathways, seasonal variability, and location and use of aquifers.
- Land use and ownership.
- Climate.

This information is necessary for facility design and, to some extent, for planning efficient borrow site development.

5.1.1.2.1 Indirect Methods--Indirect investigative methods include collection of existing site information and remote sensing techniques. These methods do not require drilling or excavation and are appropriate for the initial stages of site investigation.

Site investigations usually begin with compilation and review of existing information pertinent to the site. Sources of information include Soil Conservation Service County Soil Surveys, U.S. Geological Survey topographic and surficial geology maps, published literature, State geological survey information, and county records of geotechnical tests associated with previous construction projects. This information can be very useful for planning the scope and approach of further site investigation activities.

Geophysical remote sensing techniques that can be applied during site investigation include electrical survey methods, ground-penetrating radar, and seismic refraction. All of these techniques are conducted on the surface but provide information about the subsurface. The selection of geophysical techniques depends to a large degree on the geologic setting (White and Brandwein, 1982).

Electrical resistivity surveying can be used to delineate the depth of the water table as well as the presence of subsurface layers or lenses of different permeability that have contrasting resistivities (e.g., clay and sand layers). However, electrical resistivity methods cannot be applied in certain geologic settings where general subsurface resistivity is relatively high and are best used in areas (e.g., the Atlantic Coastal Plain) where electrical resistivities of subsurface materials contrast strongly (White and Brandwein, 1982). Further information on electrical surveying may be found in the U.S. Environmental Protection Agency (1978) and Freeze and Cherry (1979).

Seismic refraction surveys can give valuable information about the depth to bedrock, the subsurface bedrock topography, and the condition (fracturing) of the bedrock (Cichowicz et al., 1981). In addition, the seismic velocity of a geologic material is altered by the degree of weathering and water saturation and therefore can provide information about the variability of these parameters in the subsurface. However, because of the multitude of variables that can affect a material's characteristic seismic velocity, seismic results can be difficult to interpret, especially in areas with complex subsurface geology or in areas where there is little contrast in seismic propagation velocities in the subsurface. For this reason, limited exploratory drilling will usually be necessary in conjunction with seismic surveys to confirm interpretations based on this technique (Cichowicz et al., 1981). More detailed information on seismic refraction surveying may be found in Doorin (1960).

Ground-penetrating radar also has some utility to site investigations for locating buried structures and pipes and for indicating depth to shallow bedrock. However, it is limited by a shallow depth of penetration when compared to other techniques (White and Brandwein, 1982).

The advantage of indirect techniques during the early stages of site investigation is that their use can reduce drilling costs and costs associated with laboratory tests and analysis. Much information about a site, including an indication of its technical suitability as a containment facility site, can be gained at a relatively low cost. In addition, the information gathered indirectly can be used to plan direct site investigations, ensuring that these are carried out as efficiently and economically as possible.

5.1.1.2.2 Direct Methods--Direct methods of site investigation include drilling boreholes and wells and excavating pits and trenches. The purpose of these methods is to expose subsurface material so that the physical conditions can be observed and measured (e.g., faults, slickensides, sand seams, depth to bedrock and to the water table, penetration tests, and in situ permeability) and to obtain samples of subsurface material for laboratory testing of engineering properties.

Direct investigations are conducted during the final stages of site investigation and must provide sufficient information for input to the facility design. The scope of investigation necessary to accomplish this goal will vary from site to site according to the complexity of the subsurface geology, the potential for seasonal variability in site conditions, and the amount of information about the site that is already available.

Regulatory personnel from the Wisconsin Department of Natural Resources have recently published the following recommendations for site investigations for clay-lined landfills (Gordon et al., 1984):

- Drill an adequate number of soil borings across the site to characterize the soil deposits within and beneath the site. The borings should extend a minimum of 25 feet below the anticipated site base grade or to the water table, whichever is deeper.

- Install a sufficient number of water table observation wells and piezometers to define both the horizontal and vertical groundwater flow directions.

- Excavate backhoe pits on a grid pattern across potential clay borrow sources to characterize their depth, areal extent, and uniformity and to obtain samples of the clay material for testing.

- Perform appropriate laboratory tests on samples from the potential clay borrow sources to determine if they will meet the design specifications.

These recommendations are presented as examples; detailed exploration needs are site and facility specific.

Subsurface heterogeneities can lead to increased permeability (seepage) or loss in strength in the foundation. Where these are suspected, it may be appropriate to dig test pits and trenches to identify and determine the prevalence of these features. Downhole television monitors also can be used in boreholes to identify important subsurface features such as faults, fractures, slickensides, and zones of permeable material.

Accessible (pits, trenches, and large boreholes) and inaccessible (boreholes and wells) methods of site investigation are summarized in Tables 5-1 and 5-2. These methods are discussed in more detail in U.S. Department of Interior (1974). Methods of obtaining disturbed and undisturbed samples during both accessible and inaccessible site exploration are discussed in U.S. Department of Interior (1974) and ASTM (vol. 04.08, 1985). Methods of conducting laboratory tests of engineering properties on these samples are discussed in Chapter 3 and Appendix A of this document. Detailed discussions of general geotechnical site investigation techniques may be found in Winterkorn and Fang (1975).

Hydrogeologic site investigations are necessary for planning the groundwater monitoring system and for estimating hydraulic stresses that may act on the facility so that they may be properly considered during facility design. Further information on conducting hydrogeologic investigations and on installing monitoring wells and piezometers may be found in U.S. Environmental Protection Agency (1983), Fenn et al. (1977), Johnson Division (1975), and Lutton et al. (1983).

5.1.2 Liner Material Selection and Characterization

Soil liner materials are selected based on their ability to meet specific performance standards and the costs to bring the material onsite. Requirements that must be met for a soil to perform properly as a liner material include:

TABLE 5-1. ACCESSIBLE METHODS OF SUBSURFACE EXPLORATION

Methods	Procedure	Type of soil and in-place condition	Limitations	Use
Trenching	Excavate 3 ft min. width by hand, dragline, power shovel, bulldozer preferable; explosives if necessary; min. bracing or slope unstable soils.		Depth about 20 ft or to groundwater or unstable material.	Access for logging and disturbed sampling for laboratory test, for reconnaissance and feasibility design stage; and for hand-cut undisturbed sampling for final design or for field tests such as field density, permeability, full-sized bearing capacity tests. Unsatisfactory in unstable cohesionless soils. Economical and best method for shallow explorations of borrow, foundation, and aggregate deposits.
Cuts	Same as trenching, performed on gentle to fairly steep slopes; steps up slope may be necessary.			
Test pits	Excavate rectangular hole, 3 ft by 5 ft min. at working level, by hand or hand-operated power tools; explosives if necessary. Cribbing required over 5 ft depth. Log and sample as excavation progresses when sheeting, inclined poling, or notched-box cribbing is required in unstable soils and for groundwater control.	Coarse-grained soils, including those containing large quantities of gravel and cobbles, and soft weathered rock; and all fine grained soils, dense consolidated, wet or saturated or dry and hard; loose unconsolidated, wet or saturated and soft or dry and granular	Depth to 50 ft, infrequently 80 ft, or groundwater if pervious strata and high flow.	Use same as above, except is more expensive and used in areas of limited access by heavy equipment and for greater depths. Best method for "hand cut" undisturbed sampling except in unstable soils or below groundwater. Nonaccessible methods, Table 5-2, recommended for undisturbed sampling of fine grained unstable soils below water table.
Accessible boring	Drill 28 in. min. dia hole, using heavy power-operated disc, bucket, helical augers, single tube or core barrels in stable soils; log and sample as excavation progresses; casing required for protection during sampling and inspection.		Depth of 100 ft in soil, 150 ft in rock. Requires heavy drill rig.	Use same as above for stable soils in place of test pits; very economical if equipment is available and area is accessible.
Accessible caissons	Same as accessible borings; casing and air pressure required in unstable soils.		Same as above, used only when caisson is part of construction.	Limited use, used primarily in establishing footing grade during construction for individual caissons, under very poor foundation conditions and/or under water.
Tunnels and drifts	Excavate accessible holes, 5 by 7 ft min., using hand or hand operated power tools, lagging required.		Expensive, used only under special conditions.	Limited use, for final exploration of dam-site abutments when other methods have disclosed questionable conditions that cannot be resolved otherwise.
Blasting	Expose strata using explosives and hand or power tools.	Same as above but primarily for consolidated dry soils and bedrock.	Limited to exposed faces or outcrops.	Use to expose rock faces and outcrops for rip-rap and crushed aggregate sources and to indicate size and shape of particles that may be expected during quarrying.

Source: U.S. Department of Interior, 1974

TABLE 5-2. NONACCESSIBLE METHODS OF SUBSURFACE EXPLORATION

Methods	Procedure	Type of soil and in-place condition	Limitations	Use
Auger boring (hand)	Rotate and force auger bit into soil, withdraw and empty when full. Auger bits, 2 to 8 in., helical or post hole.	Fine-grained cohesive, fairly hard to soft or fine-grained, noncohesive, dense to loose, weakly cemented, or dry or moist; with particles ½ in. to 1½ in. depending upon size of auger.	About 20 ft, 80 ft with tripod; unsatisfactory in unstable cohesionless soils below groundwater; slow in hard soils.	(1) Advance hole. (2) Data for logging. (3) Representative disturbed samples for classification, index tests, and standard properties tests. (4) Access for field penetration and permeability tests. (5) Access for undisturbed sampling.
Auger boring (power)	Same as above, using powered drill rigs. auger bits, 4 to 24 in., helical, disc, or bucket. Over 28 in. considered to be accessible.	Fine-grained as above, and coarse-grained soils with particles as large as 3 in. depending upon auger.	Economical depth about 40 ft, over 100 ft with special equipment; unsatisfactory in unstable cohesionless soils below groundwater; slow in hard, dense soil.	Same as above.
Drive-tube boring	Force open pipe or tube, with sharpened edges, without rotation, into soil; withdraw and remove soil. Thin- or thick-wall tubing or pipe, 2 to 8 in. dia.	Fine-grained cohesive and slightly cohesive soils such as loess, firm to soft clays, and silts.	About 80 ft depending upon equipment. Not satisfactory in coarser fine-grained soils, clean sands, or cohesionless soils below water table.	Same as above.
Percussion (churn) drilling	Chopping and cutting action by impact of heavy chisel-edged bit. Water added and cuttings form slurry that is removed intermittently by pump or bailer. For holes larger than 4 in.	Coarse-grained soil containing cobbles and boulders, and hard, dense, fine-grained soils and rock.	Unsatisfactory in unstable soil or fractured rock; no information for logging or samples for classification.	Used with other methods to advance hole through hard, cemented strata, coarse gravel, boulders, or other obstructions.
Wash boring	Chopping and cutting by impact and twisting action of lightweight bit, and jetting action of circulating water to remove cuttings. For holes from 2 in. to over 8 in. dia.	Fine- or coarse-grained soils, with small amounts of gravel and few cobbles; fairly hard to soft; weakly cemented to loose; above or below water table.	No information for logging or samples for classification; slow in hard or cemented layers.	(1) Used with other methods to advance hole particularly through unstable soils requiring casing. (2) Penetrate fine-grained soils to establish depth to bedrock. (3) Drill holes for groundwater observation. (4) Provide access for sampling and penetration testing of impervious soils above groundwater or pervious or impervious soils below.

(continued)

TABLE 5-2 (continued)

Methods	Procedure	Type of soil and in-place condition	Limitations	Use
Jetting	High-velocity water jet directed downward from pipe raised and lowered in short strokes; erodes soil, which is carried upward by water. For holes 2 in. to over 10 in. dia.	Fine- or coarse-grained soils; weakly cemented noncohesive or cohesive; above or below water table.	No information for logging or samples for classification; slow in hard cohesive soils.	Same as (1), (2), and (3) for wash boring.
Rotary drilling	Power rotation of bit; cuttings removed by circulation of drilling mud or water; holes 1½ in. to over 10 in. dia.	Fine- or coarse-grained, compact or cemented soils, and rock.	No information for logging or samples for classification; difficult in loose, coarse-grained soil with cobbles and boulders.	(1) Advance hole. (2) Access for field penetration test (not suitable for well permeameter test or groundwater observation if drilling mud used) (3) Access for disturbed or undisturbed sampling.
Rotary drilling	Power rotation of bit; cuttings removed by circulation of air. Holes 2 in. to over 10 in. dia.	Fine- or coarse-grained, compact or cemented soils and rock.	Information for logging and samples for classification; unsatisfactory in loose coarse-grained soil with cobbles and boulders.	(1) Advance hole. (2) Access for field penetration test. (3) Well permeameter test. (4) Groundwater observation. (5) Access for disturbed and undisturbed sampling. (6) Advance hole to install casing for nuclear moisture-density probes.
Continuous sampling	Drive-tube boring or rotary drilling (core boring) that provides samples as a result of advancing the hole.		Depends upon the method selected.	

Source. U.S. Department of Interior, 1974.

- Low permeability (usually $\leq 1 \times 10^{-7}$ cm/s) when compacted

- Sufficient strength to support itself and the overlying facility components without failure when compacted to the required permeability and thickness

- Compatibility with waste or waste leachate to be contained (i.e., no significant loss in permeability or strength when exposed to waste or waste leachate).

Cost considerations also enter into material selection when liner material must be brought from offsite.

The in-place, native soil at the facility site is the ideal liner material from the standpoint of cost and convenience; this material will be excavated during foundation preparation and therefore does not need to be transported to the site. If the native soil is not suitable as a liner material, a suitable soil from a nearby borrow source can be utilized. Most engineers interviewed reported an economic haul distance of 8 to 10 miles for borrow clay, although haul distances of up to 25 miles were reported in some instances. When suitable soils are not available at economic distances from the facility, it may be necessary to blend an additive, such as bentonite, with the native soil to improve its performance as a liner material or to blend together local soils to achieve the proper material properties. These solutions are used in areas where suitable clay is scarce.

5.1.2.1 Native Soils--

Native soils include those obtained onsite and those obtained from nearby borrow areas. Design engineers' opinions differed over suitable soil types for a clay liner. These differences are, to some extent, due to regional variability in soil types. However, the differences are also due to the fact that soil selection based on the desirable characteristics for liner soils (low permeability, high strength, self-healing capacity, chemical compatibility, and low settlement and shrink/swell potential) involves some compromise between these characteristics. The soils with the lowest permeability and the highest flexibility and self-healing capacity (CH or fat clays, see Section 3.4 of this document, unified soil classification system) have the lowest strength and highest shrink/swell potential and are more affected by chemicals than other soils. Gravelly or sandy clays have high strength and a relatively low potential for settlement. However, they also tend to be brittle and crack when stressed, and they may be more likely to contain sandy or gravelly inclusions that could locally raise permeability.

Opinions differ on the suitability of fat (CH) clays for liners. Some consider that their tendency to wick moisture; to dry, shrink, and crack; and to expand upon moistening, combined with a higher likelihood of permeability changes when exposed to certain chemicals, makes them inherently unsuitable as liner materials. Others consider that their low permeability and high self-healing capacity makes them the preferred liner material as long as provisions are made to prevent moisture change in the liner during construction and operation and after closure. Interestingly, the location of the persons interviewed seemed to affect their opinion on the use of fat clays in liners. Engineers in regions where these clays were common were more positive about their use than engineers in regions where these clays are not common and other clay types are more readily available. However, all engineers agreed that the major factors influencing soil selection were its cost and its ability to be compacted to the required permeability.

If sufficient quantities of soil suitable for use as a liner material are not available at the facility site, a borrow source must be identified. When a borrow source is selected, routine testing procedures are used to screen various potential sites. The time and expense involved in permeability testing discourage its use for this routine screening. Design firms often estimate clay suitability (i.e., permeability and strength) based on quicker, more easily performed tests, including Atterberg limits, gradation (particle size distribution), and compaction tests (standard Proctor).

Once a potential borrow source has been identified, the site should be investigated, with the methods described in Section 5.1.1, to determine the amount of suitable soil present at the site and the degree of spatial variability of soil properties in the soil deposits and to confirm that the soil is sufficiently impermeable to serve as a liner material. Borrow source investigation results then can be used to plan an efficient extraction procedure for the liner material.

Prior to facility design, representative samples of the liner material are subjected to laboratory compaction and permeability tests to establish the relationship among moisture content, density, compactive effort, and permeability. This information is needed for preparation of the facility design and the construction specifications. Although both standard and modified Proctor compaction tests are used, standard Proctor is preferred by most engineers because it results in a wetter optimum moisture content for a given soil. Fixed-wall, flexible-wall, and consolidation cell permeameters are used to measure laboratory permeabilities and to establish the moisture content and compactive effort at which minimum permeabilities may be achieved (for further discussion of these techniques, see Chapter 3).

Strength of the liner and foundation soils also must be measured for input to the design. The triaxial compression strength test (ASTM D2850-82) is generally preferred for measuring soil strength. However, the direct shear test (ASTM D3080-72, D2573-72) or vane shear test may also be used. The shear test chosen should mimic the type of failure most likely to occur in the soil liner.

The results of compaction, permeability, and strength tests are used to establish an envelope of acceptable values for parameters that will be used during design to establish material and compaction specifications. Sufficient testing should be conducted to determine the range of variability in these soil properties and to determine the suitability of all soil types that may be encountered at the soil source. Correlating these test results with soil index properties, appearance, and feel enables ranges of acceptable values of these properties to be established and specified for routine screening of liner material as it leaves the borrow source (see Section 5.3.4).

Most design engineers interviewed also recommended that compatibility studies of liner soils with wastes or waste leachates be conducted as a part of the material selection, especially when the soils are comprised of fat clays. However, no cases were uncovered (by the authors) where compatibility problems with a natural soil resulted in rejection of that soil as a liner material. One of the biggest problems with compatibility testing is the selection of a representative waste or leachate composition. Another is deciding how much of a change in soil properties (e.g., permeability) on exposure to the waste or waste leachate constitutes incompatibility. More discussion on the techniques and the problems associated with compatibility testing may be found in Chapter 4 of this document.

5.1.2.2 Admixed Soils--

When the in situ native soil is not suitable as a clay liner and nearby borrow sources of suitable soils do not exist, it may be economical to use bentonite or other clay materials as a soil additive to decrease the permeability of the native soil. Bentonite is a clay material composed of mostly sodium-montmorillonite (with minor amounts of calcium-montmorillonite and other clay minerals) and is, as a result, highly expansive with the addition of water. Bentonite's expansive nature enables relatively small amounts (5 to 10 percent) to be added to a noncohesive soil and makes it cohesive and behave similarly to a soil containing 50 percent nonbentonite clay (Kozicki and Heenan, 1983). Blending two native soils can also be used to produce a soil with the desired material characteristics.

Important parameters to consider in the design of bentonite liners include aggregate particle size of the dry bentonite, mineralogy (as reflected by swelling potential), and the required bentonite application rate. Both granular and milled (powdered) bentonite are currently used as admixed liner material. However, a Canadian firm with extensive experience in designing bentonite liners recommends that only powdered bentonite be used in liners (Kozicki, Ground Engineering, Ltd., Regina, Saskatchewan, personal communication, 1984) because powdered bentonite mixes uniformly throughout the soil mass, making intimate contact with the soil grains. Granular bentonite cannot be as intimately mixed with the soil. A failure of one bentonite/sand admixed liner has been attributed to the use of granular bentonite (Diamond, 1979).

The type of exchangeable cations present in a bentonite influences its ability to lower the permeability of a native soil and is an important property to control. Sodium is the predominant cation in high-swelling bentonite; its high-swelling capacity minimizes the amount that must be used to lower the permeability of a soil. Other bentonites can contain significant amounts of exchangeable calcium, resulting in a clay that has a lower swelling capacity. Thus, it is important to select and specify a bentonite that has the degree of swelling desired. This can be done easily with a simple test (Rollins, 1969).

The amount of bentonite to admix with the native soil to achieve a specified compacted permeability varies according to soil conditions. In general, 3 to 8 percent bentonite will lower the permeability of most granular material to between 1 × 10^{-7} cm/s to 1 × 10^{-9} cm/s (Kozicki and Heenan, 1983). However, it is necessary to determine the optimum application rate and moisture content for the specific soil/bentonite admixture. Usually this is accomplished by conducting a series of compacted permeability tests on admixtures with different percentages of bentonite. Alternatively, one design engineer determines the proper percentage by the amount necessary to bring the liquid limit of the soil to 45 or to produce a CL classified soil (Pacey, Emcon Associates, Inc., San Jose, California, personal communication, 1984). Once the proper percentage of bentonite is determined, density, moisture content, compactive effort, and permeability relationships need to be established for facility design purposes. Proper soil percentages for blended soil liners can also be determined through the above procedures.

All personnel experienced with bentonite additives agree that compatibility tests are especially critical for admixed liners. The high-swelling clay minerals are generally more affected by chemicals than other clay minerals. As an example, sodium-montmorillonite is easily changed to calcium-montmorillonite when it undergoes ion exchange with solutions high in calcium salts. This change will seriously reduce the swelling potential of bentonite and thereby increase the permeability of the admixture.

One approach to dealing with the compatibility problem is to pretreat the admixed bentonite with the waste liquid it is to contact. This method is currently being used for bentonite admixed linings for brine ponds. When brine is used to wet the bentonite admixtures, all chemical effects take place during installation, precluding any change once the brine pond is filled (Kozicki, Ground Engineering, Ltd., Regina, Saskatchewan, personal communication, 1984). Although it may not be reasonable to pretreat bentonite with hazardous wastes or waste leachate, it may be possible to pretreat admixtures with a nonhazardous material that will have a similar effect on the bentonite.

5.1.3 Facility Design

Once the site and liner material have been selected and characterized, the design of the hazardous waste containment facility can begin. Facility design is accomplished through standard geotechnical practices but must be tailored to the individual site geology and facility operational requirements. The following text summarizes some important points about the design of clay liners gathered through our interviews and review of the literature. Other facility components (e.g., leachate collection systems and caps) necessary for proper facility performance are not covered by this document. More detailed discussions of earthwork design engineering may be found in references such as Winterkorn and Fang (1975), U.S. Department of the Navy (1982), and U.S. Department of the Interior (1974).

5.1.3.1 Configuration--

The configuration of the clay liner is determined by the configuration of the containment facility, which is determined by topography, geology, hydrogeology, land ownership, existing structures, and waste volume requirements. Generally, facilities are rectangular, but topographic constraints or land availability can result in irregular shapes. Facility size is usually determined by projected waste volume requirements and planned modes of facility operation. However, facility size also can be limited by technical considerations, such as seismic design criteria.

Facilities may be excavated below ground, built above ground and contained by dikes, or built partially above and below ground. Generally the designs of clay liners for these types of facilities are similar, except that dike design is required for aboveground facilities and groundwater control measures are usually required for facilities sited below the water table (intergradient design).

5.1.3.2 Foundation Design--

Foundations for clay liners are designed to control settlement and seepage and to provide structural support for the liner. The natural foundation should provide satisfactory contact with the overlying liner, minimize differential settlements and thereby prevent cracking of the liner, and provide an additional barrier to leachate migration from the facility.

5.1.3.2.1 Settlement--Settlement is usually not a problem for clay liner foundations. Most clay liners are sufficiently thick to withstand some differential settlement of the foundation soils. As long as the topography is fairly uniform and significant soil heterogeneities are not present, differential settlement should be minimal. However, several design engineers recommend excavating and recompacting the upper 1 to 2 feet of foundation soil to control local settlement and seepage prior to liner installation. Several engineers also recommend that foundation settlement analysis based on the site's soil properties (determined during site investigation) be conducted during the design of the facility. These analyses should take into account the weight of all facility components on the foundations, especially footings for pile-type structures such as leachate collection risers, which, if improperly designed, can be forced into or through the liner. Compensated foundation, which implies that the weight of soil extracted from the site balances the weight of fill material, also can be used to minimize subgrade settlement (Vesilind et al., 1983). Techniques for conducting settlement analyses are given in any standard soil mechanics text.

Haxo (1980) notes that differential settlement is a localized structural stress phenomenon; therefore, the greater the liner's thickness and elasticity, the greater the tolerance range for differential settlement. A sufficiently thick liner can engage in self-healing if the subgrade settles nonuniformly.

The ability to predict the extent of settlement depends upon the type of process anticipated to cause settlement. Primary consolidation, which is a reduction in void ratio due to removal of pore fluids by mechanical loading, generally occurs according to the consolidation theory developed for soil. Basically, the theory states that the rate and amount of compression is equal to the rate and amount of pore fluids squeezed out of the soil (Anderson, 1982).

Secondary consolidation (densification) depends upon the applied load and the chemical and physical nature of the solid particles and the waste. Therefore, secondary consolidation is more irregular and less predictable than primary consolidation and may be significant in settlement of plastic clay soils, heterogeneous fill materials, organic materials, and other compressible materials.

Tertiary consolidation (densification) occurs when the volume of solids is reduced. The effect of tertiary consolidation on mineral soils is minimal; however, it may be a major concern with organic soils, organic waste, soluble materials, and materials subject to chemical attack. Tertiary consolidation is highly irregular and is influenced by a number of environmental factors that make it difficult, if not impossible, to predict (Anderson, 1982).

5.1.3.2.2 Seepage--Seepage both into and out of the facility must be controlled during construction and site operation. Although the clay and flexible membrane liners are designed to accomplish this goal, for their optimum performance and because it may function as a backup liner, the foundation also should be designed to control seepage. For intergradient facilities, seepage can reactivate slickensides in the foundation soil. If these features are present near the toe of sidewall slopes, slope failures can result (Boutwell, Soil Testing Engineers, Inc., Baton Rouge, Louisiana, personal communication, 1984). Opening of slickensides or joints can occur from stress removal by excavation and when soils heave in response to unbalanced water pressures in underlying permeable strata (Boutwell and Donald, 1982).

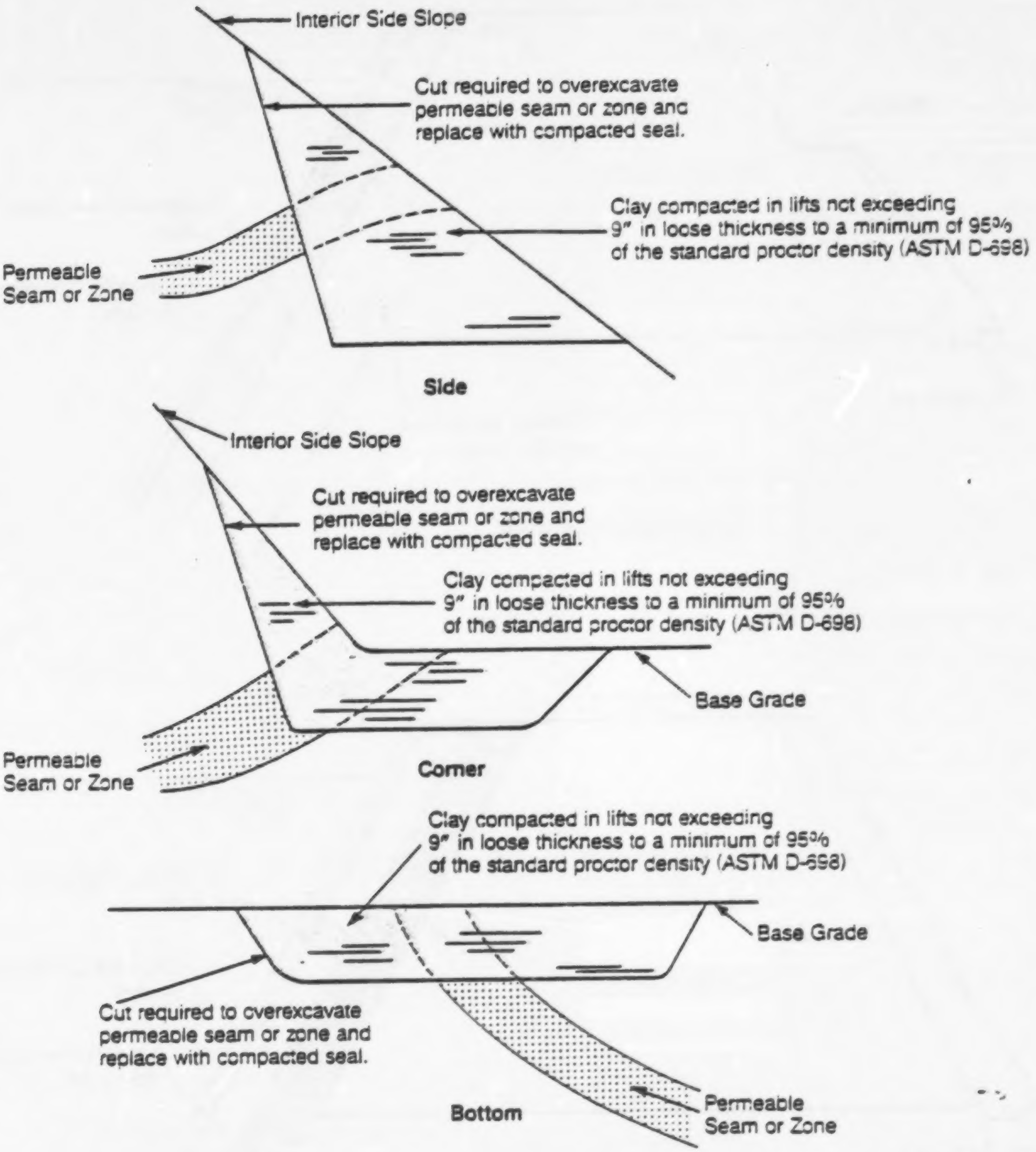
Heterogeneities such as large cracks, sand lenses, or sand seams in the foundation offer pathways for leachate migration and could cause piping failures. Soft spots in the foundation can cause differential settlement, possibly causing cracks in the liner and damage to the leachate collection and leak detection system. Cracks and sand lenses or seams also can cause problems during liner construction if the facility penetrates the water table. An illustration of this problem was provided by a design engineer during one of the interviews. In the case he cited, a 2-foot clay liner was installed without adequate site investigation or foundation preparation. A small fracture in the foundation base was connected to an artesian (geopressured) aquifer 30 feet below the landfill bottom. The overburden that was removed during construction originally provided confining pressure on the fracture. After the liner was installed, hydrostatic pressure in the crack exceeded the confining pressure provided by the liner, causing a rupture of the liner. The expense of excavating the ruptured liner and crack, grouting the crack, and pumping the aquifer to reduce the hydraulic head could have been avoided or minimized if adequate site investigation and foundation preparation had been practiced prior to liner installation.

Solutions to these problems include the various dewatering systems (e.g., pumping wells, slurry walls, trenching, and pumping) to lower the hydraulic gradient on the facility (Boutwell et al., 1980). Further discussion on dewatering methods may be found in Cashman and Haws (1970), and Section 5.1.3.4.7 describes intergradient facility design in more detail. Other methods to control foundation seepage include properly keying dikes into the foundation subsoil, selecting impermeable materials for dike cores, grouting cracks and fissures in the foundation soil with bentonite or other grouting material, and designing compacted clay cutoff seals to be emplaced in areas of the foundation where lenses or seams of permeable soil occur. Figure 5-1 illustrates typical clay cutoff seal designs.

5.1.3.2.3 Dike Design--Containment facilities built above ground or partially above ground require dikes to support the aboveground portion. Hazardous waste containment facilities may be designed to be above ground for several reasons. Regulations may require a certain distance between the facility bottom and the nearest underlying aquifer or, in some cases, the water table. Aboveground design may be necessary to achieve this separation. In regions with high water tables, limits placed on excavation depth in order to prevent bottom heaving or rupture may necessitate a partial aboveground design (see Section 5.3.4.1.7). In addition, low-cost, suitable dike construction material may be readily available at or near the facility site, making aboveground construction more economical than excavated facilities.

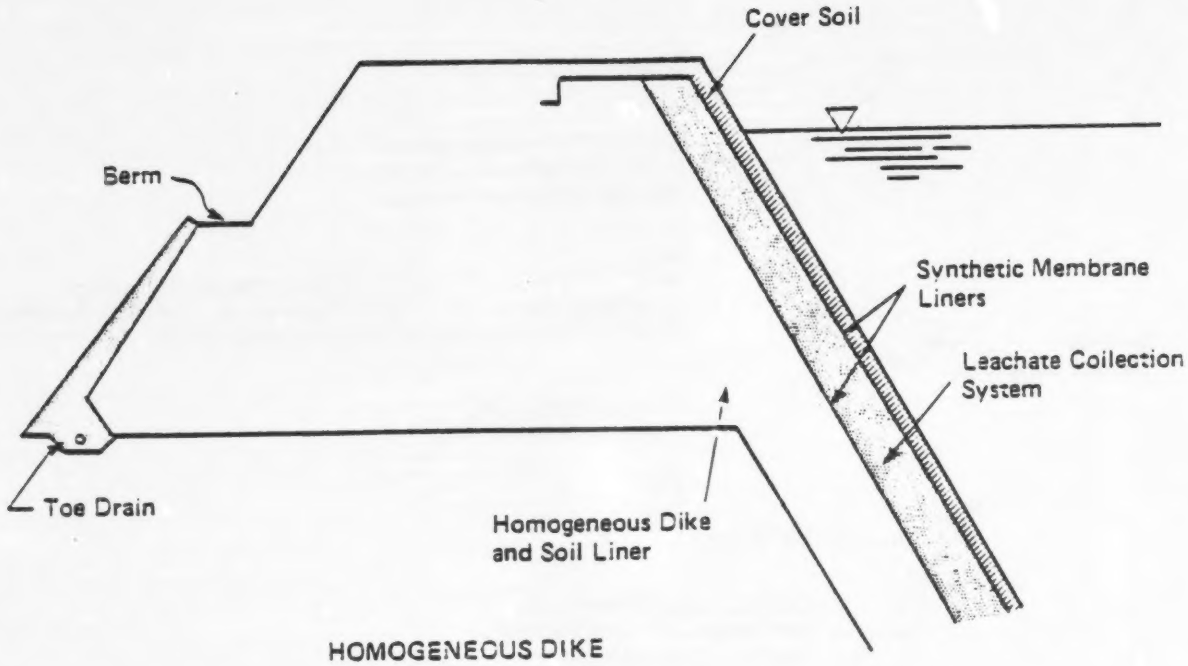
A containment facility dike serves as a retaining wall to resist the lateral forces imposed by the stored wastes. Design for retaining characteristics requires slope stability analysis, which is normally accomplished by using the Bishop method of slices (Boutwell et al., 1980). As the waste produces much of the outward force on the dike, the geotechnical properties of the wastes must be defined. In addition, time-related changes in the properties of both the waste and the dike material resulting from consolidation, settlement, changes in saturation, or chemical interactions must also be evaluated (Boutwell et al., 1980). Figure 5-2 illustrates typical dike configurations.

Erosion resistance and control of desiccation must also be considered during the design of dikes. Berms and vegetation may be used to control erosion. In arid regions special designs incorporating gravel-filled troughs in the dike crest have been used to provide a method to prevent desiccation cracking. If the trough is kept filled with water, the exposed upper portion of the dike can be kept moist. For further information on dike design, the reader is referred to Sherard et al. (1963), U.S. Bureau of Reclamation (1973), Winterkorn and Fang (1975), U.S. Department of the Army (1977), and U.S. Department of the Navy (1982).

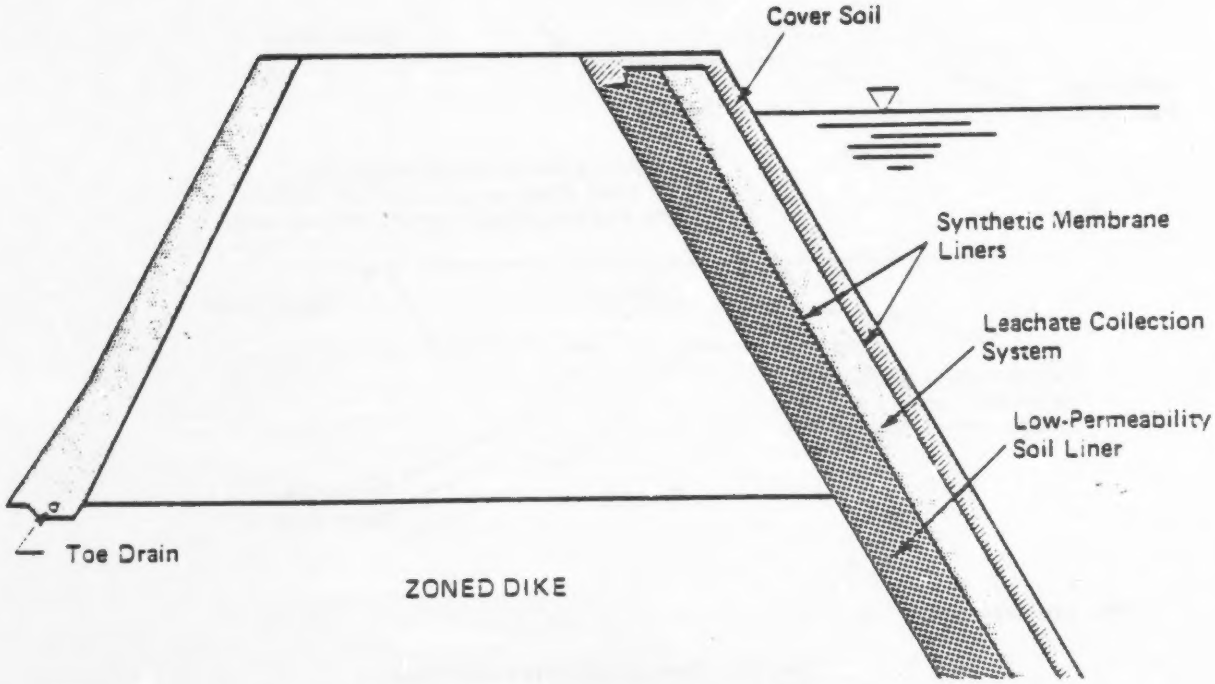

Interior Side Slope
Cut required to overexcavate permeable seam or zone and replace with compacted seal.
Clay compacted in lifts not exceeding 9" in loose thickness to a minimum of 95% of the standard proctor density (ASTM D-698)
Permeable Seam or Zone
Side
Interior Side Slope
Cut required to overexcavate permeable seam or zone and replace with compacted seal.
Clay compacted in lifts not exceeding 9" in loose thickness to a minimum of 95% of the standard proctor density (ASTM D-698)
Base Grade
Permeable Seam or Zone
Corner
Clay compacted in lifts not exceeding 9" in loose thickness to a minimum of 95% of the standard proctor density (ASTM D-698)
Base Grade
Cut required to overexcavate permeable seam or zone and replace with compacted seal.
Permeable Seam or Zone
Bottom

After Waste Management, Inc.

Figure 5-1. Compacted clay cutoff seal.


Cover Soil
Berm
Synthetic Membrane Liners
Leachate Collection System
Toe Drain
Homogeneous Dike and Soil Liner

HOMOGENEOUS DIKE


Cover Soil
Synthetic Membrane Liners
Leachate Collection System
Low-Permeability Soil Liner
Toe Drain

ZONED DIKE

Figure 5-2. Dike components and typical configurations.

5.1.3.2.4 Sidewall Design--Sidewall slopes of the hazardous waste containment facilities identified during this project ranged from 4 to 1 (horizontal to vertical) to vertical. Factors that influenced the selection of sidewall slopes included waste volume/landfill area considerations, foundation and liner soil stability, equipment operation constraints, and stability of other facility components.

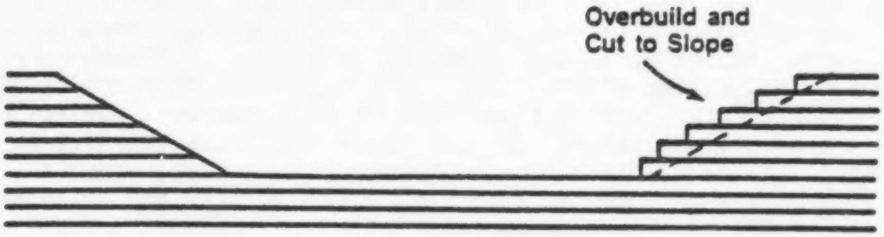
One factor that influences the choice of sidewall slope is whether the sidewall liner is to be compacted in horizontal lifts or in continuous lifts parallel to the liner surface (Figure 5-3). Generally, the continuous lift method cannot be used for side slopes steeper than around 2.5 to 1 because of operational limitations of compaction equipment. However, this limitation varies according to soil characteristics and equipment type. In one case, described during an interview, continous lifts were compacted on slopes as steep as 2 to 1 with a sheepsfoot roller. However, the facility was small, the slopes were short, and the roller had to be both pushed and towed to negotiate the slope. In soils common to Louisiana, a bulldozer can operate on slopes as steep as 2 to 1 but tends to tear the liner material. A slope of 2.5 to 1 is recommended for bulldozer operation, and a slope of 2.8 to 1 is recommended when sheepsfoot rollers are used (Boutwell and Donald, 1982).

Some design engineers interviewed preferred horizontally compacted side slopes to those with continuous lifts because the former are more stable and they allow steeper slopes and hence greater facility waste capacity. However, most engineers interviewed believe that, for horizontal sidewall lifts, the orientation of lift boundaries and the compacted clay fabric perpendicular to the liner surface increase the likelihood of seepage through the liner, limiting the desirability of horizontally compacted sidewall slopes. Thus, most engineers consider it especially critical to ensure that lifts are adequately tied together in horizontally compacted side slopes. However, no case studies were found during this investigation, which demonstrated increased seepage due to horizontal sidewall lift compaction.

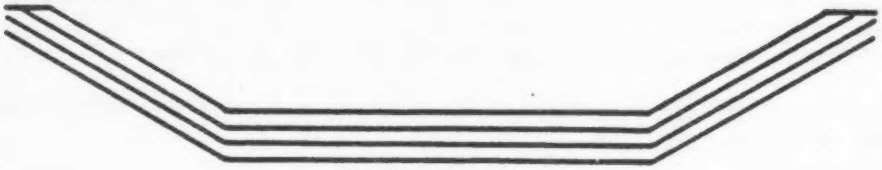
Other sidewall slope considerations collected during our interviews and literature review include:

- Flexible membrane liner (FML) manufacturers generally recommend that, when an FML is part of the liner system, sidewalls should not be steeper than 3 to 1. Tracked vehicles placing earth materials on FMLs tend to stall, spin their tracks through the loose earth, and damage the FML on steeper slopes (Morrison et al., 1982). However, FML-lined facilities with side slopes of 1 to 1 were encountered during our survey. Opinions differed among design engineers on this subject.

- For admixed bentonite liners, slopes of 3 (H) to 1 (V) are generally preferred. Mechanical spreading methods can be used on 2.5-to-1 slopes, but this is the marginal case. Hand placement can be used on slopes of 2 to 1, with mixing and compaction performed by towing equipment up and down the slope (Kozicki and Heenan, 1983).



Horizontal Lifts



Continuous Lifts

Figure 5-3. Methods of liner sidewall compaction.

- The angle of repose of sand corresponds to approximately a 2-to-1 slope. Thus, if a granular leachate collection system is designed to extend up the slopes, sidewall slopes cannot exceed 2 to 1.

- Although steep sidewalls with horizontal lifts maximize volume and provide greater stability, construction costs can be greater than with continuous lifts because of logistic and scheduling requirements and the extra liner material that must be used and then trimmed away.

- With thinner linings (2 to 3 feet), extra width to accommodate equipment may result in the sidewall lining being thicker than the bottom lining for horizontally compacted sidewalls.

- One engineering firm preferred horizontally compacted sidewalls for thick linings ($\geq$5 feet) and continous sidewall liner lifts for thin linings ($\leq$2 feet).

- One engineer recommended that 4-inch lifts (versus 6-inch bottom lifts) be used in horizontally compacted liner sidewalls.

- Slopes of 3 to 1 or less tend to collect water if not properly smoothed.

- Maximum sidewall slopes of 2 to 1 are advisable for liners composed of highly plastic soils because of the loss of stability of these soils when they are saturated (Day, 1970). Highly plastic soils contain high amounts of smectite clay minerals. The high dispersivity of these minerals results in a loss of strength upon wetting.

Slope stability analysis must be conducted for both foundation and liner soils to ensure that shearing stresses developing within sidewall slopes following excavation or dike construction will not exceed the available shear strength of the soil and cause a failure of the slope. The shear strength (or cohesion) of the soil and its variability in the soil mass, degree of saturation of the soil (or pore water pressure), slope height and inclination, heterogeneities in the soil mass, and expected stresses on the slope are all important inputs to slope stability analyses. Applicable strength tests for the waste and soil include a measure of relative shear strength by a cone penetrometer, vane shear tests, remolding index, drained direct shear, triaxial compression, and consolidation tests. Strength tests should be selected to mimic the expected mode of failure in the soil (Haxo, 1983).

Several methods of slope stability analysis are currently used. Classic limit equilibrium methods are generally used for earthwork design.

These methods take a free body from the slope and, using estimates of the forces acting on the body, calculate the equilibrium shear resistance of the soil and compare it to the shear strength of the soil to indicate the factor of safety (Fang, 1975). Regardless of the specific procedure used for carrying out the computations, the following principles are common to all limit equilibrium methods: a potential failure surface is postulated; the shearing resistance required to equilibrate the assumed failure mechanism (i.e., the shear stresses required along the failure surface to balance the driving forces of the unsupported slope) is calculated by means of statics; the calculated shearing resistance required to maintain equilibrium is compared to the available shear strength of the soil along the assumed failure plane (the ratio of the existing strength to induced shear stress is one definition of a factor of safety); and the mechanism with the lowest factor of safety is found by iteration.

Choosing the method depends to some extent on the properties and hydrologic conditions of the soils to be evaluated. For homogeneous soils with equal pore pressure distributions, methods that consider the whole free body are appropriate, such as the friction circle method (Taylor, 1948). If soil properties vary through the soil mass, the method of slices, which divides the free body into several vertical slices, is appropriate (Bishop, 1955; Fellenius, 1927). If straight line failure planes can be identified in the soil mass, the wedge method can be used (Lambe and Whitman, 1969). The wedge method has been recommended for evaluating the stability of clay liners placed on side slopes (Boutwell and Donald, 1982). For facilities below the water table it is necessary to consider pore water pressure, and the effective stress (total stress minus pore water pressure) should be used instead of total stress (Fang, 1975). Methods of stability analysis that consider these factors include the Bishop and Morgenstern method (Bishop and Morgenstern, 1960) and the Spencer method (Spencer, 1967). Factors that must be considered when slope stability is calculated in special problem soils are presented in Table 5-3.

Essentially two types of slope stability problems occur in clay: short-term stability (end-of-construction case) and long-term stability (steady-seepage case). The short-term case applies just after the excavation is completed and assumes that time has been insufficient for any water to move in or out of any representative soil element within the soil profile. When a soil profile is believed to be relatively homogeneous and without discontinuities, this is a reasonable assumption and has been used with much success. Should the clay have substantial joints and fissures, drainage may occur so quickly along these discontinuities that the problem may not be adequately represented by the assumption (Esu, 1966). In general, only short-term stability analysis is necessary for interior containment facility slopes if the facility is to be filled shortly after construction; long-term slope stability should be considered for the outer slopes of aboveground facilities (Boutwell and Donald, 1982).

A factor of safety is customarily calculated during slope stability analysis. The factor of safety is calculated as a ratio of terms reflecting conditions that impact a slope's stability. Terms that are used for

TABLE 5-3. FACTORS CONTROLLING STABILITY OF SLOPED CUT IN SOME PROBLEM SOILS

SOIL TYPE	PRIMARY CONSIDERATIONS FOR SLOPE DESIGN
Stiff-fissured Clays and Shales	Field shear resistance may be less than suggested by laboratory tests. Slope failures may occur progressively and shear strengths reduced to residual values compatible with relatively large deformations. Some case histories suggest that the long-term performance is controlled by the residual friction angle which for some shales may be as low as 12°. The most reliable design procedure would involve the use of local experience and recorded observations.
Loess and Other Collapsible Soils	Strong potential for collapse and erosion of relatively dry material upon wetting. Slopes in loess are frequently more stable when cut vertical to prevent infiltration. Benches at intervals can be used to reduce effective slope angles. Evaluate potential for collapse as described in DM 7.1, Chapter 1. (See DM-7.3, Chapter 3 for further guidance.)
Residual Soils	Significant local variations in properties can be expected depending on the weathering profile from parent rock. Guidance based on recorded observation provides prudent basis for design.
Sensitive Clays	Considerable loss of strength upon remolding generated by natural or man-made disturbance. Use analyses based on unconsolidated undrained tests or field vane tests.
Talus	Talus is characterized by loose aggregation of rock that accumulates at the foot of rock cliffs. Stable slopes are commonly between 1-1/4 to 1-3/4 horizontal to 1 vertical. Instability is associated with abundance of water, mostly when snow is melting.
Loose Sands	May settle under blasting vibration, or liquify, settle, and lose strength if saturated. Also prone to erosion and piping.

Source: U.S. Department of the Navy, 1982.

these calculations include available shear strength versus required shear strength, required soil cohesion versus available cohesion, actual friction angle versus stable friction angle, actual height versus stable height, and resisting moments versus moments tending to cause failure. Probabilistic statistical methods also have been developed for factor of safety calculations (Fang, 1975).

A factor of safety of 1.0 means that if the slope is constructed perfectly as designed, it will not fail. However, because actual construction is rarely perfect, a larger safety factor is usually used to design slopes for critical facilities. For public earth works projects (e.g., dams) in California, a safety factor of 2.0 is used. One design engineer considered a safety factor of 1.7 to be the minimum acceptable for use on outer slopes of hazardous waste facilities (Reynolds, California Department of Health Services, Sacramento, California, personal communication, 1984).

Static slope stability analysis is appropriate in areas with little or no seismic risk. Dynamic slope stability must be considered in areas where significant ground motion can occur (Fang, 1975; also see Section 5.1.3.4.6).

In summary, the following guidance should be applied to the design of slopes (Lutton et al., 1979):

- Examine, sample, and test foundation conditions to ensure that it is not weak and likely to participate in displacement.
- Conduct detailed engineering stability analyses for any site where the consequences of slope failure are serious. Estimate changes in hydrology, seismic stability, identify average and worst case patterns, and integrate all information into a factor of safety.
- When selecting soils, consider soil shear strength, allowing for compaction and the corresponding strengthening effect.
- Specify slope inclination; decreasing the design inclination effectively increases the stability of soil slopes.
- Use underdrains, toe drains, cutoffs, and leachate collection and disposal systems for seepage control.
- Allow for freeze/thaw and dry/soak conditions in the selection of a sufficiently thick side slope soil.
- Compact soil as specified, using field tests for quality control or quality assurance. Prescribe lift thickness.
- Consider other miscellaneous factors including toe protection (e.g., from flooding conditions) the use of berms or systems of berms rather than having a single unbroken inclination, and the use of reinforcement to strengthen embankments.

5.1.3.2.5 Bottom Design--The bottom of the containment facility must be shaped to facilitate leachate collection in landfills and drainage in surface impoundments and to prevent puddling and ponding on the liner during construction. Most design engineers use a 1- to 2-percent slope for the facility base, with 2 percent being preferred.

Kmet et al. (1981) used an analytical model of landfill leakage developed by Wong (1977) to evaluate several landfill design parameters, including clay liner bottom slope. In general, increasing the bottom slope decreased the liner leakage by providing more rapid movement of leachate to the collection sump. Leakage rapidly increased when bottom slopes decreased to below 2 percent. Bottom slope increase to 5 percent provided additional reduction in leakage, with little additional benefit for steeper slopes.

The configuration of the bottom slope depends on the facility configuration and the leachate collection system design. Most designs incorporate a sand/gravel drainage layer with a system of pipes, with the liner bottom sloping to the pipes and the pipes and underlying liner sloping to a collection sump. Based on the analytical model described previously (Wong, 1977), Gordon et al. (1984) recommend a maximum leachate flow distance to the pipe network of 150 feet, and 50 feet has been stated as the reasonable minimum due to construction practicalities (Kmet et al., 1981). Small facilities may have a pipeless collection system, and one design engineer interviewed preferred a properly designed granular drainage layer without pipes to a piped system because it precludes the necessity of cleaning out pipes.

5.1.3.3 Liner Design--

Clay liners are constructed of compacted clay soils installed in a series of lifts of specified thickness. The liner must be sufficiently thick and impermeable to retard leachate flow and to provide structural support to overlying facility components. For clay liners, a permeability of 1×10^{-7} cm/s is required by Federal regulations. One design engineer recommended that a permeability of 1×10^{-8} cm/s be specified to provide a factor of safety.

Liner thicknesses of 1 to 12 feet of native clay were encountered during the course of this study, although most design engineers recommended 5 to 10 feet of clay. State regulations usually determine liner thickness, with 2 feet (as recommended by EPA guidance) as the minimum. Transit time prediction methods (described in Chapter 8 of this document) also have been used to specify liner thickness.

In general, the liner is designed to be uniformly thick over the entire facility except for thicker areas that will be excavated for the leachate collection sump and for any leachate collection pipes recessed into the landfill bottom. This is necessary so that, following excavation, the liner will have the uniform, specified thickness over the entire landfill bottom as illustrated in Figure 5-4. In addition to this, engineers working for the Department of Natural Resources, State of Wisconsin, recommend an extra foot of liner under leachate collection lines (Gordon et al., 1984).

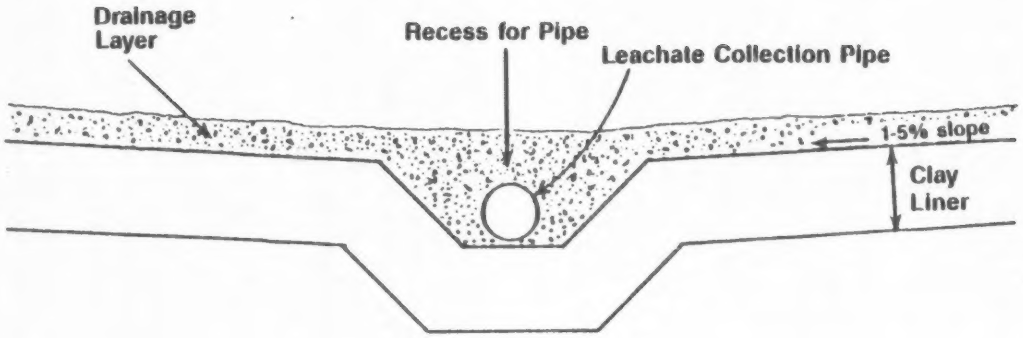


Figure 5-4. Liner design for collection system pipes and sump.

Extra liner thickness and compactive effort have been recommended for the toes of sidewall slopes to combat seepage and to ensure that the bottom and sidewall liners are adequately tied together.

Lift thickness is selected by the design engineers and the contractors depending upon soil characteristics, the compaction equipment to be used, and the required compactive effort. Lift thickness is usually specified in the construction specifications. The liner lifts must be thin enough so that adequate compactive effort reaches the lower portion of the lift. Emplacement of thin lifts ensures that the entire lift is adequately compacted and because more additional compactive effort is transferred to lifts below the lift being installed. However, these advantages must be balanced with the added construction expenses associated with thinner lifts since more lifts must be compacted to achieve the specified liner thickness. Interviews with design engineers indicate a general preference for loose lift thickness of 6 to 9 inches. However, loose lift thicknesses of up to 15 inches have been encountered during our survey, and a 2-foot lift was used over a sand drainage layer at one facility (see Section 5.1.3.3). Some engineers think that adequate compaction can be achieved with thick lifts if heavy enough compaction equipment is used. Four-inch lifts were recommended by one engineer for horizontally compacted sidewalls.

Clay liners may be designed to be installed over the entire facility (small facilities) or in segments (large facilities and continuous-operation facilities). If the liner is installed in segments, a beveled or step-cut joint between segments (Figure 5-5) should be specified to ensure that they are properly tied together. A bevel with an 8 to 1 slope was specified for a large facility where the liner was installed in wide transverse strips (Boutwell, Soil Testing Engineers, Baton Rouge, Louisiana, personal communication, 1984).

Most admixed bentonite liners are only 4 to 6 inches thick. Bentonite company literature indicates that liner thickness can be as small as 4 to 6 inches because of the low permeability (10^{-9}) that can be achieved with bentonite (IMC, 1982). However, Federal guidance recommends a 2-foot clay liner, and many States require liners up to 10 feet thick. Regulatory personnel in some of these States did not think that thin bentonite linings satisfy the regulations. Besides failing to meet the thickness requirements, thin liners may not provide adequate structural stability for the overlying facility components such as leachate collection risers and may be hard to construct to uniform thickness. In general, clay liner thickness and permeability requirements must be independently satisfied; the use of a clay with a lower permeability than required does not justify a thinner liner (Boutwell and Donald, 1982).

Two facilities with bentonite liners are included in the case studies section of Chapter 7. At Site O, local soils were augmented with 3 percent bentonite to achieve the required permeability. The actual permeability achieved was 8.3×10^{-8}; however, a 4-foot liner was still used at the site. In addition, 3 percent lime was added to stabilize the bentonite by reducing its swelling capacity (by replacing sodium with calcium in the

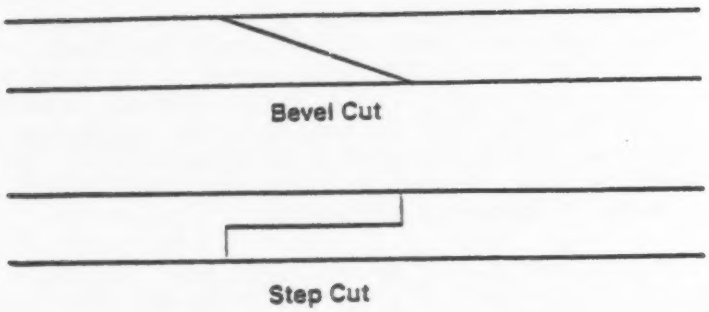


Figure 5-5. Methods of keying-in liner segments.

bentonite). The same effect and permeability could have been achieved without lime addition by using lower cost bentonite already with a high-calcium-montmorillonite content.

Site P presents an approach to achieving both low permeability and high strength while minimizing the addition of bentonite. In this double-clay-lined site, the upper liner consists of a 1-foot compacted layer of admixed polymer bentonite and native soil (5×10^{-8} cm/s) sandwiched between two layers (3 ft above and 1 ft below) of recompacted native soil of a higher permeability (10^{-6} cm/s). The lower liner is composed of 6 inches of compacted admixed liner underlain by 5 feet of recompacted or in situ native soil. The sides are single lined with 18 inches of compacted native soil overlying 6 inches of compacted admixed soil. The designers of the site used the recompacted native soil to provide structural strength and protection to the admixed liner with the admixed liner providing lower permeability. It is not certain whether this liner design would satisfy current regulatory requirements, especially in States requiring thick liners.

Liner system configuration differs according to the type of containment facility. In landfills, the clay liner usually lies below a synthetic liner that lies below the leachate collection system. In some cases, the clay liner may be sandwiched between two synthetic liners or vice versa. Leak detection systems may be installed between the liners or under the liner system. A leak detection system may consist of a sand, gravel, and/or geotextile layer underlying the entire facility (continuous coverage) or individual lysimeters or other instruments at specific points under or between liners (discrete coverage). Collection lines from the leak detection system either pass through the clay liner or pass under the liner to exit at the landfill periphery. The latter design is preferred by many engineers because objects penetrating a liner offer potential pathways for leakage through the liner.

The relationship of the clay liner to other system components for a waste pile is the same as for a hazardous waste landfill, except that a primary synthetic liner is not required and thus may not be present in a waste pile. The leachate collection system will rest directly on a clay liner when it is the primary liner.

The major difference between the liner system in an impoundment and that of a landfill or waste pile is that because impoundments by definition are designed for holding bulk liquids, leachate collection systems are not installed on top of the liner. Surface impoundment liner system designs incorporating clay liners can consist of a single clay liner or may consist of a system of redundant liners (all clay or clay and synthetic) with leak detection or collection between or under the liners. If the wastes are to remain in the impoundment after closure, synthetic linings are required. Outlet pipes from leak detection/collection systems (below the primary liner) may pass through the primary liner or may run under it to the periphery of the facility.

A rip-rap layer on the upper side slopes of surface impoundments may be necessary to protect against wave erosion (Section 5.1.3.4.2). A discharge structure for loading wastes into the pond will prevent scouring of the liner surface (Section 5.1.3.4.2). Adequate free board (sidewalls above the liquid surface) to contain runoff from large rain storms should be included in surface impoundment designs (U.S. Environmental Protection Agency, 1982b).

When clay liners are installed over granular leak detection layers, the granular layer often does not provide enough stability for compacting the first lift of liner material. Techniques that allow equipment to 'bridge' over sand layers without causing rearrangement and damage include:

- The use of a geotextile over the drainage layer (this also prevents piping)
- The use of lighter compaction equipment that can 'bridge' the sand
- The specification of thicker lower lifts.

At one large facility, this problem was solved by using a loose lift thickness of 2 feet for the first lift over the sand layer. The normal lift thickness at this site was 15 inches; the use of large compaction equipment enabled thick lifts to be specified.

5.1.3.4. Special Design Considerations--

This section describes some special design practices that are needed to prevent specific causes of clay liner failure. A full discussion of each of these failure mechanisms, along with illustrative case studies, may be found in Chapter 6 of this document.

5.1.3.4.1 Control of Erosion--Erosion can be a problem during liner construction and on dikes and freeboard areas in completed facilities. The most accurate soil loss prediction tool that is now field-operational is the U.S. Department of Agriculture's Universal Soil Loss Equation (USLE) (Wischmeier and Smith, 1978). This equation has been used for agricultural erosion control planning for more than a decade. The maximum erosion rate should not exceed 2 tons per acre using the USLE (U.S. Environmental Protection Agency, 1982b). Important design considerations for erosion control are listed below (Lutton et al., 1979):

- Erosion-resistant soils (erosion resistance is quantified by the soil erodibility factor, K, in the USLE) should be selected.
- Dispersive clays and soils in a dispersed condition should be protected from erosion or avoided in facility areas where erosion can occur.

- The overall landfill configuration is important. The two factors, L (slope-length factor) and S (slope-steepness factor), in the USLE dominate the erosion aspects of runoff.
- It is generally essential to divert natural drainage from outside the immediate site (through external runoff diversion) to prevent erosion in the facility excavation.
- Final dike slopes more than 5 feet high should be protected from erosion by building berms and gutters along the top and sides of the slope.
- Vegetation should be started as soon as possible on exterior dike walls and be well maintained throughout the life of the site.
- The use of soil covers is often specified to minimize exposure and to protect otherwise bare soil at the construction site. Applications should always be encouraged for intervals when construction is interrupted.
- A program of long-term maintenance can be used to avoid erosion problems after closure of the landfill site. This is especially important for flood-retaining structures for facilities sited in the 100-year flood plain.

5.1.3.4.2 Control of Scouring--Scouring is the erosion of the liner or sidewalls of a containment facility by the force of moving water. Surface impoundments often need to be protected from wave erosion on their upper side slopes. Rip-rap (loose rock) is the most widely used material for this purpose. Concrete aprons can be used, but studies by the U.S. Department of Interior have shown that rip-rap is generally more effective and easily placed (Small, 1981). Rip-rap must be properly sized and placed to result in a stable protective blanket (U.S. Department of Interior, 1974). One design engineer recommends that a wind rose for the site and the pond fetch should be used to determine maximum wave height. The rip-rap layer can then be designed, based on wave height and expected fluctuations of liquid levels in the pond. Rip-rap generally is uniform in size, and dumped rip-rap performs better than hand-placed rip-rap. More information on the design of rip-rap layers may be found in U.S. Department of the Interior (1974) and U.S. Environmental Protection Agency (1982b).

Clay liners in surface impoundments must also be protected from erosion by discharge of waste into the impoundment. At least one clay liner failure identified during this study was attributed to erosion from discharging wastes into the pond. Discharge structures used to reduce this erosion include concrete or rip-rap aprons, discharge tubes with upward facing outlets, and various weirs, such as a reverse duckbill weir (Day, 1970).

Johnson and Cole (1976) reported that a bentonite liner in a papermill lagoon was protected from scouring by covering with coarse-grained material. First the liner was covered with 3 to 4 inches of till, and then a 6-inch gravel cover was placed over the till in the most susceptible areas (e.g., at the base of the aerators). Sidewalls with 3-to-1 slopes were protected from scour by 6 inches of crushed stone that was covered with rip-rap. No problems have resulted in this facility. Johnson and Geisel (1979) reported that a clay liner used in a municipal wastewater treatment lagoon was protected by covering with gravel.

Voigts and Savage (1974) described a wastewater treatment lagoon lined with natural clay. Rip-rap was added to the slopes to protect against wave action and erosion, and a concrete scour pad was placed under the aerators to prevent scouring the liner. Stone was added around the scour pad out to 35 feet as an additional safety factor to prevent erosion. The aerator supports, which penetrated the clay liner, were sealed with neoprone water stops in the middle of the compacted clay.

5.1.3.4.3 Cold Climate Design--Failures from operations in cold climates are minimized by following design considerations outlined by Lutton (1979). Although these considerations were developed for covers, the following are relevant to liners as well:

- Evaluate soil susceptibility to undesirable frost actions and locate the earth barrier below the frost zone.
- Maintain an unfrozen soil supply.
- Consider seasonal scheduling and alternating the choice of liner soils depending on the availability of unfrozen materials.

In addition, temporary liner covers of soil or organic mulch have been used for protection from freezing temperatures

5.1.3.4.4 Control of Piping--Piping is a form of internal soil erosion that occurs below the ground surface (see Section 6.4). It occurs when fine particles migrate away from a cohesive soil layer and may result from physical causes (discordant grain size) or chemical dissolution.

Compatibility testing of the liquid waste or leachate and the liner material (see Chapter 4) is a prerequisite to minimizing piping from chemical dissolution. The compatibility testing can document the chemicals' effect on permeability. During the permeability test, observations for signs of migrating soil particles should be made and recorded. Dissolution effects (e.g., observation of an unusual color of effluent) and obvious structural changes in the soil material should also be noted. The waste and liner would be considered compatible and piping from dissolution would be considered unlikely if the test showed no permeability increases, no migrating soil particles, and no other dissolution effects.

Four laboratory tests to determine soil susceptibility to dispersive erosion have been developed by the U.S. Soil Conservation Service (SCS). Dispersion tests are a specialized group of tests for characterizing fine-grained soils suspected of having a tendency to erode rapidly. A major conclusion of a recent symposium on soil piping was that these four tests should be performed on soils where piping would cause unacceptable damage (Sherard and Decker, 1977). The four tests are the pinhole test, a test of dissolved salts in the pore water, the SCS dispersion test, and the crumb test. The test methods and extensive test data are available in ASTM Special Technical Publication No. 623.

When a clay liner overlies a granular drainage layer, control of piping can be accomplished by the incorporation of a filter layer below the clay layer. A filter layer will have a grain size slightly larger than the clay and will capture the migrating clay particles. For example, clay over gravel represents the joining of discordant grain sizes such that the clay particles can penetrate to the voids of the gravel. When a filter is placed under the clay, internal erosion is largely eliminated. In addition, the filter would help to minimize drainage problems in the sand or gravel layer (or the leachate collection system) that occur from clogging by the fine clay particles.

Filter layers may be composed of geotextiles or a layer of graded material. Resource Conservation and Recovery Act (RCRA) guidance documents have been developed by EPA, and these documents contain criteria for selecting filter layers to stop migrating particles and to avoid plugging of drainage layers (U.S. Environmental Protection Agency, 1982b). The criteria were developed by the U.S. Army Corps of Engineers and are based on grain-size ratios for the adjacent layers.

5.1.3.4.5 Control of Desiccation--Desiccation cracks are prevented by several testing, design, and construction procedures. Waste compatibility with the liner should be confirmed to avoid cracking from chemical attack. If the waste leachate creates cracks or channels in the liner material, alternative liner materials should be evaluated. The liner material should be tested to determine the liquid, plastic, and shrinkage limits. These criteria may be used to evaluate a soil's cracking potential (Lutton et al., 1979). Tables 5-4 and 5-5 show the expected volume changes associated with these indices. Soils with high volume changes have a greater tendency to crack with decreased moisture.

Clay liners may be subject to developing desiccation cracks during and immediately after installation. The clay may be protected from desiccation after construction by installing a synthetic membrane; by installing 1 to 2 feet of soil; or for surface impoundments, by putting liquids into the impoundment immediately after construction.

5.1.3.4.6 Seismic Design--Earthquakes occur across the United States. Although they are less frequent in eastern States, geological conditions are such that damage from ground motion is more widespread in the East than in California when earthquakes do occur (see Section 6.8). If a facility

TABLE 5-4. RELATIVE VOLUME CHANGE OF A SOIL AS INDICATED BY PLASTICITY INDEX AND OTHER PARAMETERS

Likelihood of volume change with changes in moisture	Plasticity index		Shrinkage limit
	Arid regions	Humid regions	
Little	0 to 15	0 to 30	12 or more
Little to moderate	15 to 30	30 to 50	10 to 12
Moderate to severe	30 or more	50 or more	10 and less

TABLE 5-5. SOIL VOLUME CHANGE AS INDICATED BY LIQUID LIMIT AND GRAIN SIZE

Passing No. 200 sieve (%)	Liquid limit (%)	Probable expansion (%)
>95	>60	>10
60 to 95	40 to 60	3 to 10
<30	<30	<1

Source: Lutton et al., 1979.

is located in an area likely to experience ground motion from seismic events, it should be designed to withstand this ground motion. The most likely types of damage from seismic events include (LARG, 1982):

- Failure of structures from ground shaking
- Failure of facility components due to soil liquefaction, liquefaction-induced settlement and landsliding, and soil slope failure in foundations and embankments
- Failure of facility components due to fault rupture
- Landsliding and collapse of surrounding structures.

Of these failures, current Federal locational standards protect only against fault rupture by requiring a 200-foot setback from active faults. The other three types of failure are caused by earthquake-induced ground motion.

LARG (1982) found that ground motion is much more important as a failure mechanism than fault rupture. More sites are impacted by ground motion than by surface faulting for a given seismic event because the only sites impacted by faulting are those actually located on a surface fault trace. LARG's study of six hazardous waste facilities in California also found that ground-motion-induced failure of tanks and poorly designed surface impoundments were potentially the most significant contributors to hazardous waste release during earthquakes.

The process of designing earthquake-resistant structures may be divided into four steps. They are:

- Determining the maximum credible or maximum probable earthquake for the site
- Determining the expected peak ground acceleration at the site from the maximum earthquake, based on regional and site-specific geologic factors
- Determining site-specific seismic hazards, such as potential for soil liquefaction, slope failure, and landslides
- Designing the facility to withstand peak ground acceleration.

The current California Water Resources Control Board hazardous waste regulations encompass these steps in addition to requiring a 200-foot setback from active Holocene faults. Specifically, the California Administrative Code (CAC) requires the following for new hazardous waste facilities or expansion of existing facilities:

- A determination of the expected peak ground acceleration at the waste management unit associated with maximum credible earthquake (CAC Article 9, Section 2595)

- Consideration of regional and local seismic conditions and faulting and site-specific surface and subsurface conditions in the above determination (CAC Article 9, Section 2595)

- Use of the peak ground acceleration to determine the stability of and safety factors for all embankments, cut slopes, and associated fills during the design life of the unit (CAC Article 9, Section 2595)

- Design of the waste management unit to withstand the maximum credible earthquake without damage to the structures that control leachate, surface drainage, erosion, and gas (CAC Article 4, Section 2547).

Determining the maximum expected seismic event and the resultant seismic loading at the site are the key elements in seismic risk analysis. The two approaches to this determination are deterministic and probabilistic (Bernreuter and Chung, 1984). The first step in both methods is to delineate regional (tectonic province) and local (fault) sources. For the deterministic method, the next step is to select the governing earthquake or maximum credible earthquake, usually the most damaging historical earthquake associated with the site. Attenuation of the seismic energy with distance from the source is then determined based on regional and local subsurface conditions. The maximum (peak) ground acceleration at the site is then determined based on surface and subsurface site characteristics.

The probabilistic approach involves developing an earthquake recurrence model for each source that could impact the site. The ground motion at the site from different earthquakes at different distances is calculated (based on local and site-specific subsurface geology), and this information is used to calculate the probability that a given level of ground motion will not be exceeded within a specified time period. One weakness of the probabilistic approach is that it results in the same probability of occurrence of a large earthquake for any time period. In the real world, the longer the time period since the last earthquake, the more likely it is that another earthquake will occur (Bernreuter and Chung, 1984). In California, the deterministic approach is considered to be the conservative or worst case approach and is required for determining peak ground acceleration at hazardous waste facilities and for large public work projects, e.g., dams (Reynolds, California Department of Health Services, Sacramento, California, personal communication, 1984).

In order to adequately estimate peak ground acceleration associated with the maximum credible or maximum probable earthquake at a site, it is very important to conduct comprehensive assessments of the regional geology and adequate site-specific subsurface investigations. Site-specific subsurface geology determines the magnitude and direction of propagation of seismic energy to such an extent that it is impossible to generalize attenuation of seismic energy with distance for seismic events. In addition, determining a site's geology is necessary for identifying features that are vulnerable to seismic ground motion such as unstable soil or rock slopes prone to landslides and unconsolidated, saturated deposits prone to liquefaction-induced settlement and failure. Liquefaction is primarily controlled by the character of ground motion, soil type, soil moisture content, and in situ soil stress conditions. Slopes most vulnerable to earthquake shocks are:

- Very steep slopes of weak, fractured, and brittle rocks or unsaturated loess that are vulnerable to transient shocks due to the opening of tension cracks
- Loose, saturated sand that may be liquefied by shocks with sudden collapse of structure and flow slides
- Sensitive cohesive soils with natural moisture exceeding the liquid limit
- Dry, cohesionless material on a slope at the angle of repose that will respond to seismic shock by shallow sloughing and slight flattening of the slope (U.S. Department of the Navy, 1983).

Estimates of ground motion are derived from peak bedrock acceleration at a site. The methodology used to determine peak bedrock acceleration should take into account regional and local seismicity, surface and subsurface geology, seismotectonic features, faulting mechanisms (e.g., thrust and strike-slip) and regional attenuation. This peak bedrock acceleration is then modified for site-specific surface and subsurface conditions that focus or attenuate seismic energy or that are sensitive to ground motion. These conditions include type, thickness, and density of materials overlying the bedrock, surface topography, and depth to groundwater (for liquefaction assessment). The goal of this approach is to define peak ground acceleration and ground motion response spectra at a site (Reynolds, California Department of Health Services, Sacramento, California, personal communication, 1984). Peak ground acceleration is then used for the seismic design of the facility. Methods for estimating earthquake ground motions may be found in Hays (1980).

Dynamic side-slope stability may be calculated through standard methods such as those described by Makdisi and Seed (1978), Sherard (1967), and Seed (1975). Computer programs are also available for designing earthworks to withstand seismic events (EERC, 1984). The risk of densification and settlement in response to ground shaking can be reduced by compacting soils to densities high enough to prevent further settlement. The potential for liquefaction can be reduced by removing and replacing sensitive (granular) soils or by lowering the water table (Eagling, 1983).

In general, seismic design of earthworks involves building structures with more strength, density, mass, and thickness. Deeper foundations, greater cross-sectional area, and better materials are specified for seismic-resistant designs. Size and configuration are also important; generally, smaller facilities are more resistant to damage from ground shaking. A safety factor (Section 5.1.2.3.4) also must be incorporated into the design (Reynolds, California Department of Health Services, Sacramento, California, personal communication, 1984).

5.1.3.4.7 Intergradient Facility Design--The construction of sites below the water table (intergradient facilities) presents problems due to seepage and hydraulic forces on the compacted clay liner. Excavations below the water table can experience hydraulically induced side-slope slippage and heave or rupture of the foundation base. In addition, a buildup of water pressure behind a recompacted clay liner can threaten the structural integrity of the liner. Although not allowed in some States (e.g., New York), intergradient facilities are constructed in other States (e.g., Louisiana) where sites with low-permeability soils above the water table are rare.

When the excavation extends below the groundwater table, it is necessary to consider the long-term or steady-seepage case for slope stability analysis. For this case, pore pressures are assumed to be in equilibrium and are determined from considerations of steady seepage (generally from the construction of a flow net or finite element analysis); the excess pore pressures generated due to the total stress changes in the slope during the excavation are assumed to have dissipated. This case is analogous to the drained-shear test, and effective stress parameters should be used. A number of analytical techniques are available that are appropriate for intergradient design. Summaries of the available total stress and effective stress methods that describe the particular assumptions on which the methods are based are presented in a number of foundation engineering texts and technical publications (e.g., Winterkorn and Fang, 1975; Schuster and Krizek, 1978).

If the clay soil in which the containment facility is excavated is located above an underlying permeable stratum (aquifer), there is potential for heave or rupture of the floor of the excavation. To prevent this from occurring, the downward pressure at the interface of the clay soil layer and the aquifer must exceed the upward hydrostatic pressure. Where the width of the excavation floor is narrow compared to the thickness of the clay soil layer over the aquifer, the shear strength of the soil can add substantially to the calculated stability of the soil layer. However, for large floor dimensions with respect to clay layer thickness or for long-term considerations, the overburden weight should exceed the upward hydrostatic force.

A related but distinctly different problem involves the stability of the compacted soil liner. Assuming it is properly designed and installed, the liner may be expected to have a nearly uniform low permeability. Water pressure can build up behind the clay liner sidewalls or under the clay liner on the excavation floor, cause it to separate from the parent soil, and cause it to heave or rupture. Whether a liner heaves or ruptures depends on the plasticity of the soil, with more plastic soils tending to heave and stiffer soils tending to rupture. The analysis of this problem involves, again, a comparison between the "uplift" water pressure and total overburden stress due to the weight of the liner (soil density and liner thickness) and any waste material in place.

Control of these problems may be achieved by at least three methods: (1) decrease the depth of excavation into the clay layer, (2) decrease the hydrostatic pressure acting on the bottom of the clay layer, or (3) increase the overburden pressure. Assuming the design depth is an economically based decision, the best permanent solution is the added weight of the waste material. However, in the short term the use of some type of dewatering system or soil grouting may be necessary if the desired depth is too deep and results in excessive uplift pressures. If the imbalance of forces is small and it is estimated that the time period between excavation of the overburden and replacement by the waste material is short, a method for evaluating the rate of heave proposed by Boutwell and Donald (1982) can be applied. They note that the uplift of the bottom of the excavation must be small with respect to the effective diameter of the excavation and that the worst case occurs when the underlying permeable stratum is very thick. The authors indicate that in some cases the analysis may show additional dewatering measures to be unnecessary.

Dewatering methods that can be used to reduce hydraulic head on the liner include dewatering wells, slurry wall cutoffs, and sump pumping. Further information on the design and applications of these dewatering processes may be found in Cashman and Haws (1970).

5.1.4 Construction Specifications and CQA Plan

These documents establish the lines of communication among the contractor, design engineer, owner-operator, and regulatory personnel and are critical to the proper construction of a clay liner. Both specifications and the CQA plan must be proposed prior to the construction. These documents usually must be approved by the permitting agency prior to construction and must be complete and detailed enough to assure them that the liner will be properly constructed.

5.1.4.1 Construction Specifications--

The two types of construction specifications are:

- Performance, which specifies a required level of performance for the completed facility components (e.g., the liner will have a permeability of 1×10^{-8} cm/s).

- Method, which specifies methodology and equipment to be used in constructing the liner.

In general, engineers designing clay liners for hazardous waste facilities prefer a combination of method and performance specifications. The performance must be specified to ensure that the liner performs as required. Most designers consider some method specifications necessary, especially if the contractor is not experienced in constructing clay liners for hazardous waste facilities and may not be aware of the importance of operational methodology in constructing clay liners so that they will not fail. In addition, moisture content, density, and compactive effort must be controlled in the field if the specified permeability is to be achieved; specification of these parameters helps ensure that it is achieved. Combination method and performance specifications must be very carefully drawn to solve the problem of the specified method that does not yield the specified performance.

Equipment (e.g., sheepsfoot rollers for compaction) and lift thickness are usually specified, and maximum clod size and scarification between lifts is sometimes specified. Prior to liner construction most design engineers specify the construction of a test fill with the same materials, equipment, and methodology that is to be used for the liner construction. Test fills can give the constructor valuable experience with the equipment, methodology, and soil to be used during construction and can help convince the constructor of the necessity of using certain equipment or methodology. Test fills are also necessary to ensure that the permeability measured in the laboratory can be achieved in the field with the equipment to be used in constructing the liner. Test fills are further discussed in Section 5.3.4.1.

Design tolerances are usually present in the specifications and/or the QA plan. Although statistical methods are available for determining design tolerances, most engineers use a 'rule of thumb' method based on their experience. A safety factor approach (e.g., specifying an order of magnitude lower permeability than required) is one approach. Another is to allow a certain percentage of test failures based on experience during quality control (QC) activities. Specifications usually include design drawings and text and for clay liners often include the following:

- Facility configuration and size
- Foundation preparation
- Liner material characteristics (e.g., index properties)
- Liner thickness and permeability
- Sidewall slope
- Bottom slope and configuration
- Lift orientation on sidewalls
- Lift thickness
- Maximum clod size
- Percent Proctor density
- Percent wet of optimum moisture content
- Scarification between lifts
- Compaction equipment and number of passes
- Test-fill compaction.

5.1.4.2 CQA Plan--

The design effort does not end with the start of construction but continues until the facility is completed. The CQA plan is prepared prior to construction and establishes the lines of communication and the testing program necessary to inform the designers, owner-operator, and regulatory agencies about whether the construction process is producing a liner that performs as required by regulation. The design engineer uses the CQA and CQC results to identify unexpected problems encountered during liner construction that can necessitate changes in the original design. The CQA program also informs the designers and/or owner-operator whether the construction specifications are being followed by the contractor. Further discussion of CQA and of the CQA plan may be found in Section 5.3 of this document.

5.1.5 Design Case Studies

This section presents, as examples of current practices, important design features of some of the clay-lined hazardous waste facilities identified during the course of this study. These examples are presented to illustrate the variability in clay liner design and construction practices. Further information on these facilities, including diagrams of the facilities, may be found in Chapter 7 of this report.

5.1.5.1 Site D--

This site, a secure landfill in the Southeastern United States, has a clay liner constructed of compacted clay from a nearby borrow area (about 3 miles away) with a maximum compacted permeability of 1×10^{-7}.* The liner is 10 feet thick and lies on a low-permeability deposit of opaline claystone or Fuller's earth. The facility is constructed below the water table, and a French drain system was installed around the site prior to excavation to drain the overlying red sand material. Side slopes are 3 to 1. A 17-foot-wide bench with a 3-foot-wide drainage trench was constructed at the top of the opal claystone. This bench enabled a thicker liner to be installed above the claystone unit. Below the bench, a minimum of 10 feet of the claystone was left as a foundation, except for the leachate collection sump, which is underlain by 4.5 feet of claystone. A test fill was specified for this facility prior to construction to check equipment performance. The clay liner was compacted in 6- to 8-inch lifts, with a sheepsfoot roller or a smooth drum vibratory roller. The clay liner is overlain by a flexible membrane liner (FML). Each landfill cell is completed prior to its operation.

5.1.5.2 Site F--

This site is a series of five clay-lined landfill cells each lined with 10 feet of recompacted clay, an FML, and a 1- to 2-foot overlying compacted clay layer to protect the FML. The clay liner was constructed of material obtained from a borrow area about 15 miles from the site. This facility was constructed above the ground with dikes to contain the wastes. The high water table at the site, regulatory considerations, and ready availability of suitable dike material (industrial slag) at the site contributed to the selection of an aboveground design. Foundation preparation included removal of industrial slag and organic silt material previously disposed on the site.

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*All permeabilities in this section are laboratory measurements unless otherwise specified.

A test fill was specified prior to construction. The liner was constructed in 6-inch lifts with a sheepsfoot roller. The sidewall slope is 2 to 1, and the sidewalls were compacted in horizontal lifts. The bottom slope of the liner ranged from 1 to 2 percent. The entire liner was installed prior to waste placement.

5.1.5.3 Site H--

This facility is a rectangular 9-acre cell used as a sanitary landfill. It is lined with a 4-foot clay liner. Prior to clay liner installation, the fill area was rough-graded. Clay was brought to the facility from a nearby borrow area. The variable nature of the borrow area required a soil's technician to be present during all removal operations to ensure that the material met the project specifications. A technician present at the landfill rechecked the material by performing the required soil tests as the clay was emplaced. The liner was compacted in 12-inch lifts with either a rubber-tired or sheepsfoot roller.

This clay liner is underlain by two 12-foot × 100-foot lysimeters for leak detection.

5.1.5.4 Site I--

This facility consists of three clay-lined surface impoundments that cover approximately 8 acres in a semiarid region of the United States. The impoundments are lined with two 5-foot compacted clay liners separated by a 15-inch granular leak detection layer. The impoundments are partly above ground and partly below ground and are contained by dikes. The dikes are 12 feet wide on top.

The interior dike sidewall slopes are 3 to 1. The soil excavated for the facility was used as the liner material. The liners were compacted in 8-inch lifts with a sheepsfoot roller. A 2-foot layer of sandy soil was placed on top of the liner for protection. Troughs for loading wastes into the impoundment were lined with rip-rap. This facility experienced failure from desiccation cracking of the liner because it was left exposed for 7 months prior to waste emplacement.

5.1.5.5 Site J--

This facility consists of six ponds and a landfill. The ponds range in size from ½ to 4 acres. The bottom liners at this facility consist of two recompacted clay layers separated by a leak detection system. The lower clay liner is 1 foot thick and the upper clay liner is a minimum of 5 feet thick. The sidewalls are constructed partially underground and partially above ground. The aboveground sections are built upon dikes. The leak-detection system is a 6-inch drainage blanket that slopes to a trench containing a 2-inch slotted polyvinyl chloride (PVC) collection pipe (Figure 7-15). The dikes around the ponds are topped with a gravel-filled trench. This trench is filled with water to help prevent desiccation of the dikes during dry months.

5.1.5.6 Site K--

This site in the Western United States consists of six double-lined ponds ranging from 1.2 to 2 acres. The pond liners were constructed of the local excavated claystone material. The liner system consists of a 1-foot recompacted basal liner. Sump and collection trenches are excavated into this bottom liner. A 1-foot sand layer covers the lower clay liner. The sand layer is overlain with 3 feet of compacted clay. The liner was compacted with a segmented smooth-wheeled roller. The facility is constructed partially in ground and partially above ground with dikes to contain the wastes.

5.1.5.7 Site L--

The landfill consists of a flat double clay liner on top of which a dike was placed to contain the wastes. The liner extends beyond the dike to form the liner for a 12-foot-wide drainage ditch that encircles the site.

The liner system at this facility consists of six layers listed below from the top down:

- An 8- to 12-inch drainage layer (sand or gravel)
- An 18- to 25-inch compacted clay layer
- A 12-inch sand leak-detection layer
- A 12-inch compacted clay layer
- A 6- to 18-inch compacted soil layer
- A bidim type C34 (synthetic soil stabilization geotextile) layer.

5.1.5.8 Site M--

This 12-acre site is designed to consist of three cells of approximately equal size. The facility is lined with 1 foot of recompacted clay. A leachate collection system is above the clay liner, and a leak detection system lies 2 feet below the clay liner. The first cell was excavated to a maximum depth of 6 feet. This provides a minimum of 5 feet of separation between the lowest layer of waste and the highest seasonal groundwater elevation. The sidewalls were excavated to a maximum slope of 3 to 1. The bottom slopes 1 percent to the center of the landfill.

The clay for the liner was obtained from a nearby borrow area. Prior to the placement of the clay liner, all large rocks, roots, and other foreign matter were removed from the foundation. The foundation was then graded to a 1-percent slope and scarified to permit better bonding between the foundation and the first clay lift. Uncompacted liner material was placed in 12-inch lifts and moistened as necessary. The clay was then compacted with "approved" equipment. Documentation of the specific types of equipment used was not available.

5.1.5.9 Site P--

The facility consists of one double-lined hazardous waste cell. The bottom liner is composed of leachate collection and detection systems as well as a series of natural soil and bentonite/soil liners. The side liner and dike containment system includes a soil and a bentonite/soil liner.

The bottom liner extends over the entire bottom and 6 feet up the sides. It is a layered system containing two drainage or collection layers and two soil liners. Proceeding from the top layer downward, the bottom liner components are as follows:

- Leachate collection system--1 foot of No. 78 gravel and sand with 4-inch perforated PVC pipes
- Upper soil barrier--5 feet of compacted soil further subdivided into three layers:
    - 3 feet of compacted native soil. Permeability = 1 × 10^{-4} cm/s.
    - 1 foot of enchanced soil, i.e., native soil blended with 9- to 12-percent polymer-treated bentonite. Permeability on the order of 5 × 10^{-8} cm/s.
    - 1 foot of compacted native soil. Permeability = 1 × 10^{-4} cm/s.
- Leak detection layer--A 1-foot sand/gravel layer with perforated pipe to detect leaks and/or to control seepage through the upper barrier. The pipes are connected to several independent monitoring stations to determine the approximate location of any leaking that might occur.
- Lower soil barrier--A 6-inch layer of enhanced soil, i.e., native soil blended with 9 to 12 percent polymer-treated bentonite. Permeability on the order of 5 × 10^{-8} cm/s.
- Buffer zone--5 feet of either in situ or recompacted native soil. Permeability = 1 × 10^{-4} cm/s.

The side liner system extends from a point 6 feet above the cell bottom to the top of the cell. This section will not have liquid impounded against it; therefore, the liner system is not as extensive as the bottom liner. Proceeding from the top layer downward, the side liner components are as follows:

- 1 foot of No. 78 gravel
- 18 inches of compacted native soil; permeability = 1×10^{-4} cm/s
- 6 inches of enhanced soil, i.e., native soil blended with 9- to 12-percent polymer-treated bentonite; permeability on the order of 5×10^{-8} cm/s.

5.1.5.10 Site Q--

The landfill consists of a single containment cell covering an area of approximately 3 acres. It is located in a former sand and gravel pit. The cell has a double liner consisting of two 4-inch layers of a bentonite/soil mixture on the bottom and side slopes up to a vertical elevation of 20 feet above the cell bottom. The bentonite/soil layers are separated by a 12-inch layer of sand on the bottom of the cell and a 6-inch layer of sand on the side slopes. The side slopes above the 20-foot vertical level are covered with a single 6-inch layer of the bentonite/soil mixture. All bottom and side slope bentonite/soil surfaces are covered with a 12-inch protective layer of gravel. The slope of the cell sidewalls varies from 2.5 to 1 to 3 to 1.

The central-plant (pugmill) mixing method was used to blend the bentonite. It was spread with a dump truck, grader, screened boards, and hand labor. A backhoe-mounted hydraulic tamper was used to compact the liner.

5.2 CLAY LINER CONSTRUCTION: METHODOLOGY AND EQUIPMENT

This section describes the methodology and the equipment that are presently used for constructing clay liners. For clarity and convenience, the discussion is broken down into preinstallation, installation, and postinstallation construction phases.

5.2.1 Preinstallation Activities

Before the liner is installed, the foundation is prepared, groundwater control measures are initiated for sites below the water table, leak detection systems may be installed, and the groundwater monitoring program is implemented.

5.2.1.1 Foundation Preparation--

The foundation of a clay-lined hazardous waste containment facility is the native soil substrate either unaltered or recompacted. For aboveground facilities, dikes constitute part of the foundation. Operations during the construction of foundations should include the following to accomplish these goals (U.S. Department of the Army, 1977):

- Stripping and excavating to remove all soft, organic, permeable, and otherwise undesirable materials. Proof-rolling with heavy equipment such as rubber-tired rollers or dozers should be done to detect soft areas likely to cause settlement.

- Filling of rock joints, clay fractures, depressions, and any areas where undesirable material has been removed. Fill material should be engineered backfill compacted to the required specifications or grouting material.

These items are important to ensure secondary containment and to ensure that the liner retains its integrity during and after construction.

Foundation construction will essentially determine the configuration of the clay liner. Whether the foundation is excavated or built above ground, the sidewall slope and bottom slope must be properly controlled and shaped. Excavation and shaping are accomplished with standard earth-moving equipment such as dozers, scraper-pans, and road graders. Slope control is achieved through traditional instrument surveying or through electronic (laser surveying) devices. Trenches are cut for the collection sump and for leachate-collection pipes if they are to be recessed into the liner.

Removal of soft spots and permeable areas in the foundation is accomplished with standard excavating equipment. Once these are removed, the resulting holes and irregularities are backfilled and compacted in a manner similar to clay liner construction. At some facilities, the entire foundation surface is disked or tilled to a depth of 1 to 2 feet and recompacted in one or two lifts. At other facilites, the foundation is left unaltered. At the end of foundation construction, the entire landfill base can be seal-rolled (rolled smooth) to seal the soil and to ensure that precipitation that may fall on the site prior to liner placement will run off properly and will not puddle or pond on the foundation surface. Final proof rolling also can be used to ensure the integrity of the finished foundation.

If the facility is constructed entirely or partially aboveground, dikes are constructed around the periphery of the excavation to serve as retaining walls for the liner (see Chapter 7 for examples of diked facilities). Dikes are generally earth or rockfill embankments and are constructed with the same techniques and equipment used to construct earth or rockfill dams. Foundations for dikes are prepared to control underseepage, to provide satisfactory contact with the overlying compacted fill, and to minimize differential settlement (U.S. Department of the Army, 1977). Dike foundation preparation operations are generally the same as described for the clay liner foundation. Dikes may be constructed in horizontal compacted lifts in a manner similar to clay liners. For further information on dike construction, the reader is referred to Sherard et al. (1963), U.S. Department of the Interior, 1974; Winterkorn and Fang, 1975; U.S. Department of the Army, 1977; and U.S. Department of the Navy, 1982.

5.2.1.2 Groundwater Control--

When a foundation excavation extends below the water table, unbalanced hydrostatic pressures from groundwater can develop in the foundation bottom and sidewalls. These pressures can cause seepage, sidewall failure and collapse, and bottom heave or rupture during subsequent facility construction and operation. Groundwater control measures can reduce this hydraulic pressure and thus reduce the likelihood that these failures will occur. The most common methods of reducing hydrostatic head around a facility are:

- Construction of a slurry cutoff wall
- Trenching and pumping
- Installation of dewatering wells for lowering the water table by pumping.

Slurry cutoff walls can be constructed by excavating a trench with a back hoe (or similar equipment) and backfilling with bentonite, asphalt, cement grout, or other suitable material. Alternately, slurry walls can be installed with the vibrating-beam technique, in which bentonite is injected along vibrating beams driven into the ground. Trench and pump methods involve digging trenches and installing sump pumps to remove infiltration. Dewatering wells are constructed by standard well-drilling techniques (Johnson Division, 1975) and are pumped to lower the water table, thereby reducing hydrostatic pressure on the facility sides and bottom.

A complete discussion of these techniques is beyond the scope of this document. For more information, the reader is referred to D'Appolonia, 1980; U.S. Environmental Protection Agency, 1984; Schmednecht and Harmston, 1980; U.S. Environmental Protection Agency, 1982a; and Cashman and Haws, 1970.

Groundwater control to preserve liner integrity is necessary only during construction and operation of the facility. Once the facility is filled, the weight of the waste will balance the inward hydrostatic forces on the liner, eliminating the need for groundwater control measures.

5.2.1.3 Leak Detection System Installation--

If a leak detection system is part of the facility design, it is necessary to install this prior to liner emplacement. This involves laying granular drainage layers and pipes over part or all of the foundation. Following its installation, the system can be covered with a properly graded soil blanket or a geotextile to prevent damage during subsequent construction activities, to provide a stable base for placement of the basal-liner lifts, and to prevent piping of clay liner material into the porous drainage layer. For further information on leak detection system design and installation, the reader is referred to the TRD on leachate collection and leak detection.

5.2.2 Clay Liner Installation

Clay liners are constructed by compacting clay soil. Clay liner materials covered in this document include natural, untreated soil and soil mixed with bentonite additives to achieve a lower permeability. Installation procedures differ for these materials, and they will be addressed separately in this document.

5.2.2.1 Natural Soil Liners--

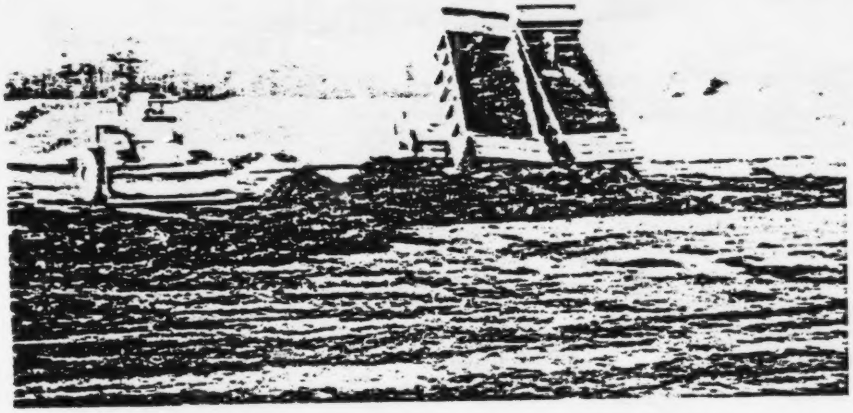
Natural soil liner material may be excavated from the site during foundation preparation or may be transported to the site from a nearby borrow source if soil material at the facility is insufficient or if the in situ soil is not a suitable liner material. QC measures are necessary at the borrow site and/or at the facility to ensure that the liner material meets design specifications (see Section 5.3.4). Excavation at the borrow site is usually accomplished with backhoes, front-end loaders, or other standard excavating equipment. One engineer recommended a side-cut excavator for use at the borrow site. This equipment is apparently very productive and produces material with a uniform, small clod size.

Prior to emplacement, a borrow pile or storage pile of the liner material usually is established at the site. Depending on climatic conditions and the condition of the clay, the borrow pile may have to be protected from moisture loss and erosion. If the clay is wet when originally placed in the pile, protection from moisture loss will reduce the amount of water that must be added to the clay prior to compaction. This is especially important in arid climates. In areas of heavy rainfall, erosion prevention may be necessary to prevent loss of liner material. Plastic or soil covers may be installed to control borrow pile moisture content and erosion. Alternatively, the pile may be graded and seal-rolled with motor graders, bulldozers, and smooth-wheeled rollers.

5.2.2.1.1 Liner Material Emplacement--Thickness requirements for clay liners for hazardous waste facilities (usually 2 to 12 feet) necessitate installing the clay in a series of lifts (layers) to ensure uniform compaction throughout the liner. For each liner lift, material from the borrow pile is emplaced into the facility with scraper-pans or trucks and uniformly distributed over the site with dozers or graders (Figure 5-6). Lift thickness is controlled during emplacement by using measuring staffs, shovel blades, or instrument surveys. Figure 5-7 shows liner material being emplaced in the area of a collection pipe. The foundation was excavated for the pipe, and, as a result, the lowest liner lift was thicker in this area. (This is also illustrated in Figures 5-4 and 7-11.)

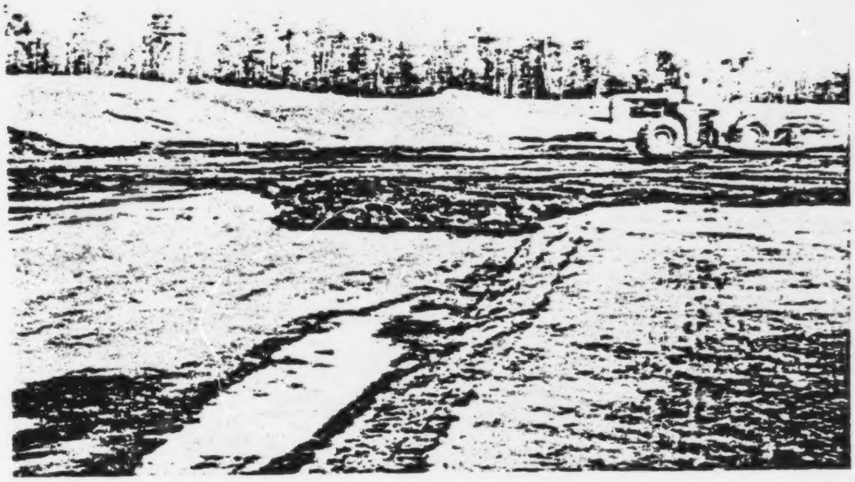
Liner emplacement methods vary with the size of the facility and the method of facility operation. For small facilities, individual liner lifts are often installed over the entire excavation, and the liner is constructed as a single unit. For larger facilities and for continuous operation facilities, where the wastes are emplaced in the facility as parts of the liner are built, the liner is installed in segments. After each liner segment is installed, it is beveled or step-cut with grading equipment so that the next segment may be tied into the previously installed segment, eliminating a potential pathway for seepage through the liner along the boundary (see Section 5.1.3.3, Figure 5-5).

5.2.2.1.2 Clod Size Reduction--Following placement, the liner material for each lift is broken up for homogenization and clod size reduction (clods are unbroken aggregates of liner material). The clod size of the liner material affects moisture control and compaction operations. Reduction in clod size increases the surface-volume ratio and decreases the time it takes for moisture to become evenly distributed within the clod (curing time), thereby facilitating moisture control operations. In addition, clod



Note: Photo courtesy of Wisconsin Dept. of Natural Resources

Figure 5-6. Liner material emplacement.



Note: Photo courtesy of Wisconsin Dept. of Natural Resources

Figure 5-7. Emplacement of liner material over foundation excavation underneath a collection pipe.

size reduction allows more effective and homogeneous distribution of compactive energy through the lift than would be achieved in lifts with clods of greater size (Withiam, D'Appolonia Consulting Engineers, personal communication, 1984; Pacey, EMCON Associates, San Jose, California, personal communication, 1984). Homogeneous compaction helps ensure homogeneous permeability.

Opinions differ among design engineers on optimum clod size for clay liner construction. Clod size recommendations gathered through interviews include 1 inch, 2 to 3 inches, no larger than one-half the lift thickness, and no larger than the lift thickness (see Table 5-8, Section 5.3.3.4). Clod size reduction is especially important in restricted areas where hand compactors must be used (e.g., around penetrating objects and in the corners of some facilities). Specifications obtained from the U.S. Department of the Navy (1982) and from the nuclear industry require that clod size be reduced to 1 to 3 inches in areas that are to be compacted by hand. Construction specifications sometimes specify maximum clod size, but not in most cases. In actual practice, clod size may not be carefully controlled. Clods with horizontal dimensions exceeding the lift thickness have been observed by the authors at some facilities under construction.

There is no documented information on the effect of clod size on the achievement of specified liner permeability in the field. However, in a laboratory study of compacted clay, Daniel (1981) demonstrated that clod size can significantly affect permeability. Table 5-6 presents the results of this experiment. Daniel (1984) has also described a field case study where large clods and inadequate curing have resulted in nonuniform moisture distribution in compacted clay liners.

The importance of clod size control depends, to some extent, on how much moisture must be removed or added to meet the specified compaction moisture content. If moisture must be added, large clods may necessitate long curing times and repeated moisture applications to ensure uniform moisture content across the clods, resulting in delays in construction schedules. Thus, efficient clod size reduction can save construction time and money.

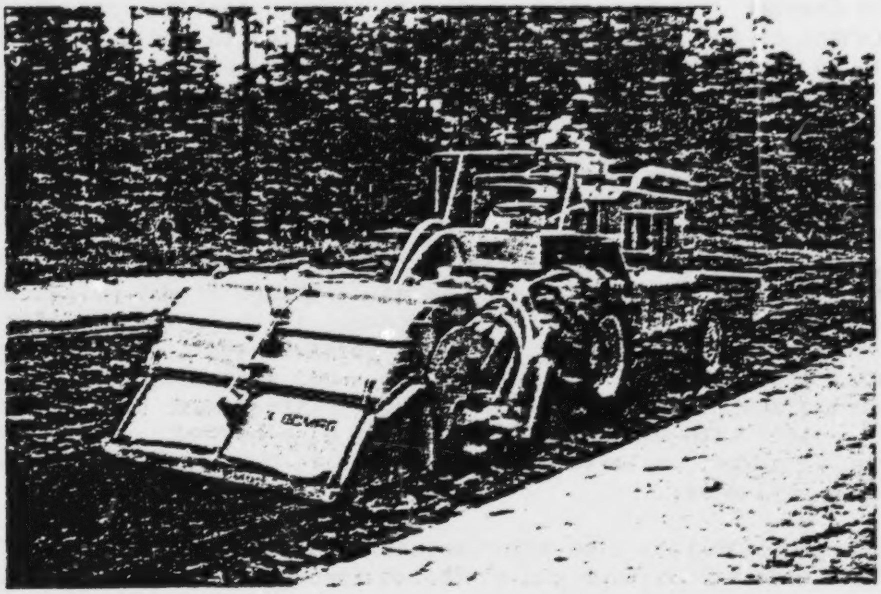
Clod size reduction is usually accomplished using disk harrows or rotary tillers with various shaped tilling blades. Availability usually determines equipment selection. High-speed pulvi-mixers (e.g., BOMAG® MPH 100), designed for breaking up soil and old asphalt pavement, have proven to be superior to tillers or disks for blending bentonite with soil (Kozicki and Heenan, 1983) and may be very good for breaking up soil clumps for untreated soil liners (Figure 5-8). The manufacturer claims that these machines are capable of reducing clod size of cohesive soils to 2 inches in a single pass and 80 percent of the clods to less than 1.5 inches after two passes.

5.2.2.1.3 Moisture Control--Moisture control includes the addition or removal of water from the liner soil to achieve the specified compaction moisture content (molding water content). Correct and uniform moisture

TABLE 5-6. EFFECT OF CLOD SIZE ON PERMEABILITY OF LABORATORY COMPACTED CLAY

Maximum size of clods (in.)	Permeability (cm/s)
3/8	2.5×10^{-7}
3/16	1.7×10^{-8}
1/16	8.5×10^{-9}

Source: Daniel, 1981.



Note: Photo courtesy of Bomag, Inc.

Figure 5-8. Use of pulvi-mixer for clod size reduction.

content is essential for compacting to the specified permeability. Minimum permeability can be obtained only if the molding water content is within several percentage points on the "wet side of optimum." (This is fully explained in Chapter 3.) For this reason, it is important to control the moisture content of the liner material carefully prior to and during liner construction to ensure that the moisture is uniformly applied and distributed throughout the soil of each lift. Nonuniform moisture distribution in clay liners has been attributed to inadequate breakup of large clods prior to compaction; uneven water distribution by sprinkling devices, especially on slopes; and inadequate curing time allowed for the water to penetrate the soil (Ghassemi et al., 1983).

Moisture is added to liner material prior to placement. Often it is most convenient to do this at the borrow area, although it may be necessary to add moisture during liner material emplacement if the soil has dried during handling and transport (Figure 5-9). Moisture addition is accomplished with sprinkler trucks, sprinkling systems, or other sprinkling devices during spreading and mixing operations. Added moisture must be thoroughly mixed into the soil with mixing devices such as disk harrows, rotary cultivators, or pulvi-mixers, to ensure that moisture is uniformly distributed throughout the soil mass.

Adequate equilibration time after moisture addition is critical to ensure that moisture is uniformly distributed throughout the soil. Clod size reduction reduces penetration time and helps achieve uniform moisture content across all clods within reasonable time. Equilibration times may reach days or weeks if soil material is very dry or if soils with a high montmorillonite content are used in the liner. Moisture addition in the borrow area may be necessary in these cases to avoid construction delays that would result if the material were moistened in place prior to compaction and then allowed to equilibrate. In dry areas or during dry periods of the year, it may be necessary to cover the soil during equilibration to prevent additional moisture loss from evaporation.

If the liner material becomes too wet to work during construction, moisture reduction may be accomplished through a combination of mechanical agitation, aeration, and solar drying.

Moisture content may be maintained during inactive periods by sloping and seal-rolling the liner to ensure proper runoff or by covering the liner with plastic or a layer of moist soil to prevent drying or overwetting of the liner material. Prevention of drying is important because desiccation can cause cracking of the liner, which can greatly increase its permeability.

Moisture content measurement is a QC activity; techniques for measuring soil moisture content are discussed in Chapter 3, and techniques for estimating soil moisture during construction are discussed in Section 5.4.3.2.3.

5.2.2.1.4 Compaction--The important theoretical aspects of compaction of fine-grained soils to achieve low permeability are discussed in Chapters 2 and 3. Compaction quality control and quality assurance are discussed

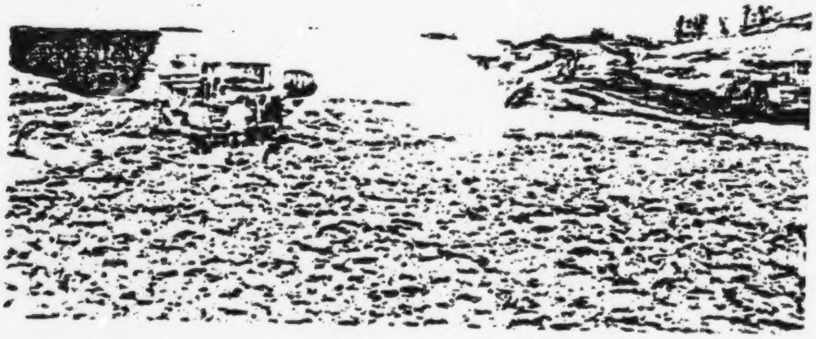


Figure 5-9. Moisture addition to liner material prior to compaction.

in Section 5.3.4. The discussion below is limited to the practical aspects of compaction in the field.

Compaction of a clay liner is accomplished through standard compaction practices as used in other earthwork construction. The following variables exist for compaction operations:

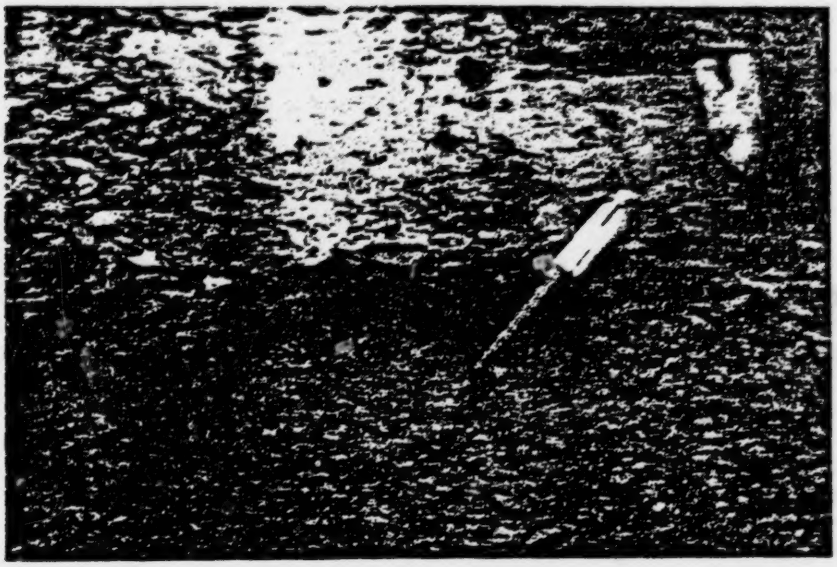
- Lift thickness and number
- Equipment type and size
- Number of equipment passes
- Soil quality
- Soil moisture content.

Because of their interrelated nature, it is necessary to control all of these variables to achieve adequate compaction in the field. Compaction of clay soils to obtain low permeability differs from compaction of earth to obtain structural stability in that compaction is performed at a higher moisture content (usually 2 to 3 percent wet of optimum). Thus, moisture content is one of the most critical factors to control when clay liners are compacted.

A critical aspect of the compaction of clay liners is to tie (join) together adjacent lifts properly. Improperly tied lifts can result in greatly increased horizontal permeability along the lift interface, an especially serious problem when sidewall lifts are compacted in a horizontal manner (see Section 5.1.3.2.4). Figure 5-10 illustrates joints and seepage along lift boundaries in sections of two experimental liners. These liners were compacted with a small self-propelled sheepsfoot roller (first figure) and a vibratory plate hand compactor (second figure). Two measures recommended by the interviewed design engineers for tying together liner lifts are (1) scarification of the surface of the last installed lift, through a disk harrow or other device, before the next lift is installed and (2) control of the moisture content of the adjacent lifts so that they are equivalent and as specified. Haxo (1983) recommended the use of compaction equipment with feet that are at least 50 percent longer than the height of the compacted lift. However, most sheepsfoot rollers have feet that are 7 to 10 inches long (Hilf, 1975), which could hamper implementing Haxo's recommendation.

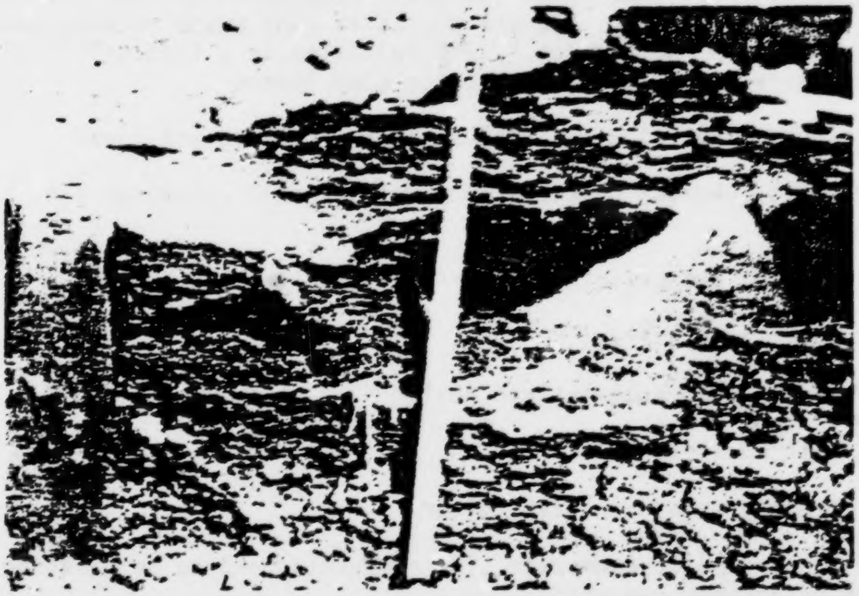
Johnson and Sallberg (1960) reported on several studies of the development of "compaction planes" or laminations between lifts. The results of these studies indicate the following:

- For all types of rollers, laminations or lift partings are much more likely to occur in soils compacted wet of optimum than in those compacted dry of optimum.



Joints Between Linerlifts

Note: Photo courtesy of Richard Warner, University of Kentucky



Joints Between Lifts Coated with Seepage

Note: Photo courtesy of Kirk Brown and Assoc., Austin, TX

Figure 5-10. Joints and seepage along lift boundaries.

- Laminations in a compacted soil are produced primarily from "springing" of the lift under compactive equipment.

- Tamping feet tend to mesh the boundary between successive layers.

Because clay liners must be compacted wet of optimum to achieve minimum permeability, this information suggests that a great deal of attention must be paid to ensure proper bonding of clay liner lifts.

Two methods of sidewall compaction are used for clay-lined hazardous waste facilities. Depending on soil conditions, if sidewall slopes are less than around 2.5 (H) to 1 (V), it may be feasible to compact the sidewalls in lifts that are continuous with the bottom liner lifts. This ensures continuity between the liner bottom and sidewalls and also orients the lift boundaries and compacted clay fabric parallel to the liner surface. However, if the sidewall slope is to be steeper than 2.5 to 1, the sidewalls may have to be compacted in horizontal lifts because most compaction equipment cannot operate on such steep slopes. When sidewalls are compacted in a horizontal manner, they are overbuilt and trimmed back to the final slope with a motor grader or excavator.

Compaction equipment is usually selected based on technical performance and availability. Some of the technical factors to be considered are whether side slopes or bottom slopes are to be compacted, lift thickness, and liner material. In some cases technical considerations are not weighed as heavily as they should be, and the choice is made based on what equipment is available that can "do the job." The following is a list of the kinds of equipment currently used for compacting clay liners:

- Sheepsfoot or clubfoot roller--self-propelled and towed

- Padfoot (pegfoot or wedgefoot) roller--self-propelled and towed

- Vibratory sheepsfoot roller--self-propelled and towed

- Rubber-tired roller

- Wobble-wheel, rubber-tired roller

- Smooth-wheeled roller

- Vibratory smooth-wheeled roller

- Vibratory plate compactor (for compaction around penetrating objects--hand operated)

- Bulldozers

- Tractors.

Table 5-7 describes the types and typical uses of compaction equipment (U.S. Department of the Navy, 1982). This table covers all types of earthwork compaction operations and is not limited to clay liner construction; only that equipment suitable for cohesive soils at wet-of-optimum moisture levels should be used for clay liners. Geotechnical textbooks and most experts interviewed recommend the use of sheepsfoot or tamping foot rollers for compacting cohesive (clay) soils to achieve low permeability. The different types of roller feet are illustrated in Figure 5-11. All of these tamping rollers are often referred to generically as sheepsfoot rollers. The general consensus is that the kneading action of these devices affects the soil fabric in a beneficial way (see Chapter 3 for a discussion of soil fabric and permeability). However, this opinion is derived from laboratory studies of impact versus kneading compaction, which show that lower permeabilities can be achieved with the latter. We know of no field studies that demonstrate the superiority of tamping or sheepsfoot rollers over other types of rollers in reducing the permeability of cohesive soils. This is an important point because laboratory kneading compactors use a tamping foot of about 0.5 inch in diameter, and the test procedure ensures that its impacts cover the entire surface area of the test soil. In contrast, the feet on a sheepsfoot roller are several inches across and are separated on the roller by a space of several inches (Figure 5-12). Thus, they do not induce shear strains as intensely or as uniformly through the soil mass as the laboratory technique does.

One engineer interviewed related a case in which the required compacted density for a clay liner could not be achieved with a sheepsfoot roller but was achieved with a vibratory smooth-wheeled roller. The differences between permeabilities for the two types of rollers were not determined in this case. An interview with another design engineer revealed that at another site (South Central United States) adequate compaction in near-saturated clays was being achieved with several passes of a bulldozer. One major engineering firm interviewed requires by specification that for cohesive soil compaction at nuclear power plants a variety of equipment (including a towed sheepsfoot roller, a self-propelled static wedgefoot roller, and a towed static wedgefoot roller) be evaluated with several different lift thicknesses in a test fill prior to construction. This facilitates selection of the most suitable equipment and lift thickness for each soil type to be compacted (see section 5.3.3.1).

Hilf (1975) mentions that sheepsfoot or clubfoot rollers are preferable to other roller types because their mixing action produces a more homogeneous liner with respect to moisture content and physical characteristics. Hilf (1975) also states that pad-type tamping foot rollers, because of larger foot end areas, do not blend and mix embankment materials as effectively as conventional sheepsfoot rollers. Figures 5-11 and 5-12 illustrate the difference between sheepsfoot and padfoot (or pegfoot) rollers.

The size and configuration of a facility may place some limits on equipment selection. Large compaction equipment (necessary for thick lift compaction) may not be usable at small sites because of large turning

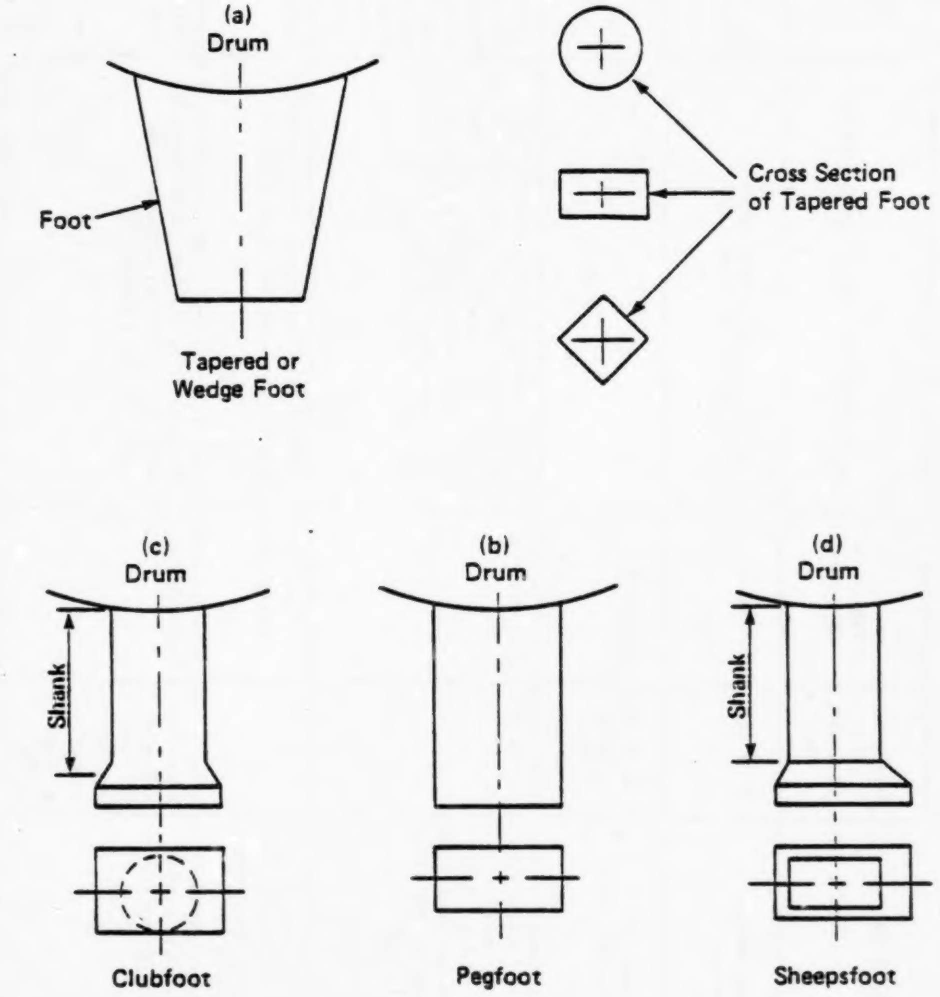
TABLE 5-7. COMPACTION EQUIPMENT AND METHODS

Equipment Type	Applicability	Requirements for Compaction of 95 to 100 Percent Standard Proctor Maximum Density			Possible Variations in Equipment
		Compacted Lift Thickness, in.	Passes or Coverages	Dimensions and Weight of Equipment	
Sheepsfoot Rollers	For fine-grained soils or dirty coarse-grained soils with more than 20 percent passing No. 200 sieve. Not suitable for clean coarse-grained soils. Particularly appropriate for compaction of impervious zone for earth dam or linings where bonding of lifts is important.	6	4 to 6 passes for fine-grained soil. 6 tp 8 passes for coarse-grained soil.	Soil Type — Foot Contact Area sq. ft. — Foot Contact Pressures psi Fine-grained soil PI>30 — 5 to 12 — 250 to 500 Fine-grained soil PI<30 — 7 to 14 — 200 to 400 Coarse-grained soil — 10 to 14 — 150 to 250 Efficient compaction of soils wet of optimum requires less contact pressure than the same soils at lower moisture contents.	For earth dam, highway and airfield work, articulated self propelled rollers are commonly used. For smaller projects, towed 40 to 60 inch drums are used. Foot contact pressure should be regulated so as to avoid shearing the soil on the third or fourth pass.
Rubber Tire Roller	For clean, coarse-grained soils with 4 to 8 percent passing the No. 200 sieve.	10	3 to 5 coverages	Tire inflation pressures of 35 to 130 psi for clean granular material or base course and subgrade compaction. Wheel load 18,000 to 25,000 lbs.	Wide variety of rubber tire compaction equipment is available. For cohesive soils, light-wheel loads, such as provided by wobble-wheel equipment, may be substituted for heavy-wheel load if lift thickness is decreased. For granular soils, large-size tires are desirable to avoid shear and rutting.
Do.......	For fine-grained soils or well graded, dirty coarse-grained soils with more than 8 percent passing the No. 200 sieve.	6 to 8	4 to 6 coverages	Tire inflation pressures in excess of 65 psi, for fine-grained soils of high plasticity. For uniform clean sands or silty fine sands, use large size tires with pressures of 40 to 50 psi.	
Smooth Wheel Rollers	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures.	8 to 12	4 coverages	Tandem type rollers for base course or subgrade compaction 10 to 15 ton weight, 300 to 500 lbs per lineal in. of width of rear roller.	3-wheel rollers obtainable in wide range of sizes. 2-wheel tandem rollers are availabe in the range of 1 to 20 ton weight. 3-Axle tandem rollers are generally used in the range of 10 to 20 tons weight. Very heavy rollers are used for proof rolling of subgrade or base course.
Do....	May be used for fine-grained soils other than in earth dams. Not suitable for clean well-graded sands or silty uniform sands.	6 to 8	6 coverages	3-wheel roller for compaction of fine-grained soil; weights from 5 to 6 tons for materials of low plasticity to 10 tons for materials of high plasticity.	

TABLE 5-7 (continued)

Equipment Type	Applicability	Requirements for Compaction of 95 to 100 Percent Standard Proctor Maximum Density			Possible Variations in Equipment
		Compacted Lift Thickness, in.	Passes or Coverages	Dimensions and Weight of Equipment	
Vibrating Sheetsfoot Rollers	For coarse-grained soils sand-gravel mixtures	8 to 12	3 to 5	1 to 20 tons ballasted weight. Dynamic force up to 20 tons.	May have either fixed or variable cyclic frequency.
Vibrating Smooth Drum Rollers	For coarse-grained soils sand-gravel mixtures - rock fills	6 to 12 (soil) to 36 (rock)	3 to 5 4 to 6	- do -	- do -
Vibrating Baseplate Compactors	For coarse-grained soils with less than about 12 percent passing No. 200 sieve. Best suited for materials with 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.	8 to 10	3 coverages	Single pads or plates should weigh no less than 200 lbs. May be used in tandem where working space is available. For clean coarse-grained soil, vibration frequency should be no less than 1,600 cycles per minute.	Vibrating pads or plates are available, hand-propelled, single or in gangs, with width of coverage from 1-1/2 to 15 ft. Various types of vibrating-drum equipment should be considered for compaction in large areas.
Crawler Tractor	Best suited for coarse-grained soils with less than 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.	6 to 10	3 to 4 coverages	Vehicle with "Standard" tracks having contact pressure not less than 10 psi.	Tractor weight up to 85 tons.
Power Tamper or Rammer	For difficult access, trench backfill. Suitable for all inorganic soils.	4 to 6 in. for silt or clay, 6 in. for coarse-grained soils.	2 coverages	30-lb minimum weight. Considerable range is tolerable, depending on materials and conditions.	Weights up to 250 lbs., foot diameter 4 to 10 in.

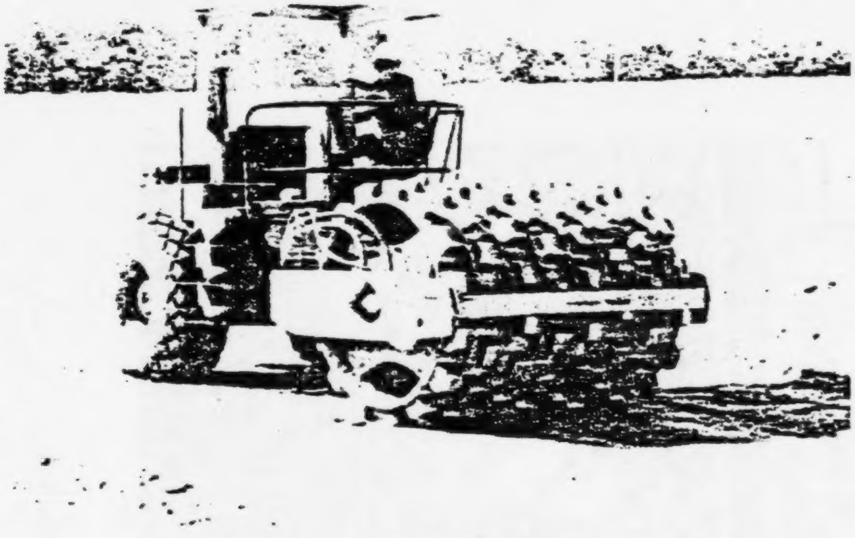
Source: U.S. Department of the Navy, 1982.



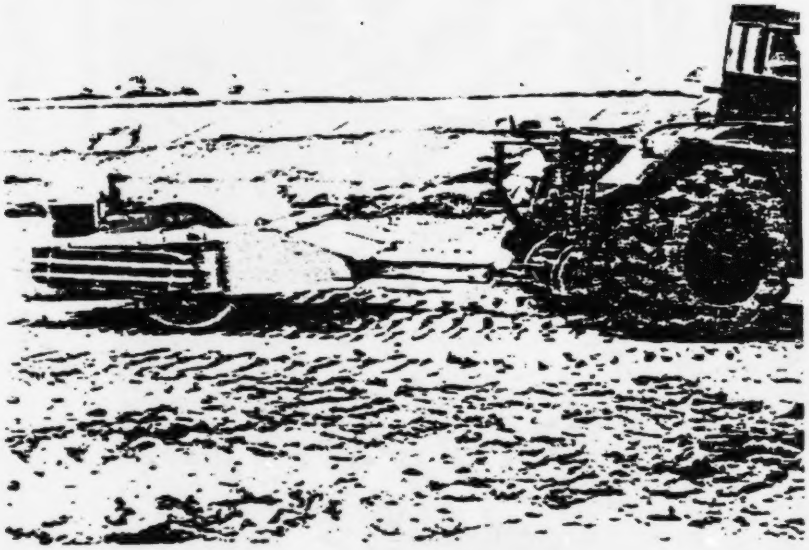
Not drawn to scale.

After Johnson and Sallberg, 1960

Figure 5-11. Sketches of different types of roller feet.



Padfoot Roller



Vibratory Smooth-Wheeled Roller

Figure 5-12. Various compacting rollers.

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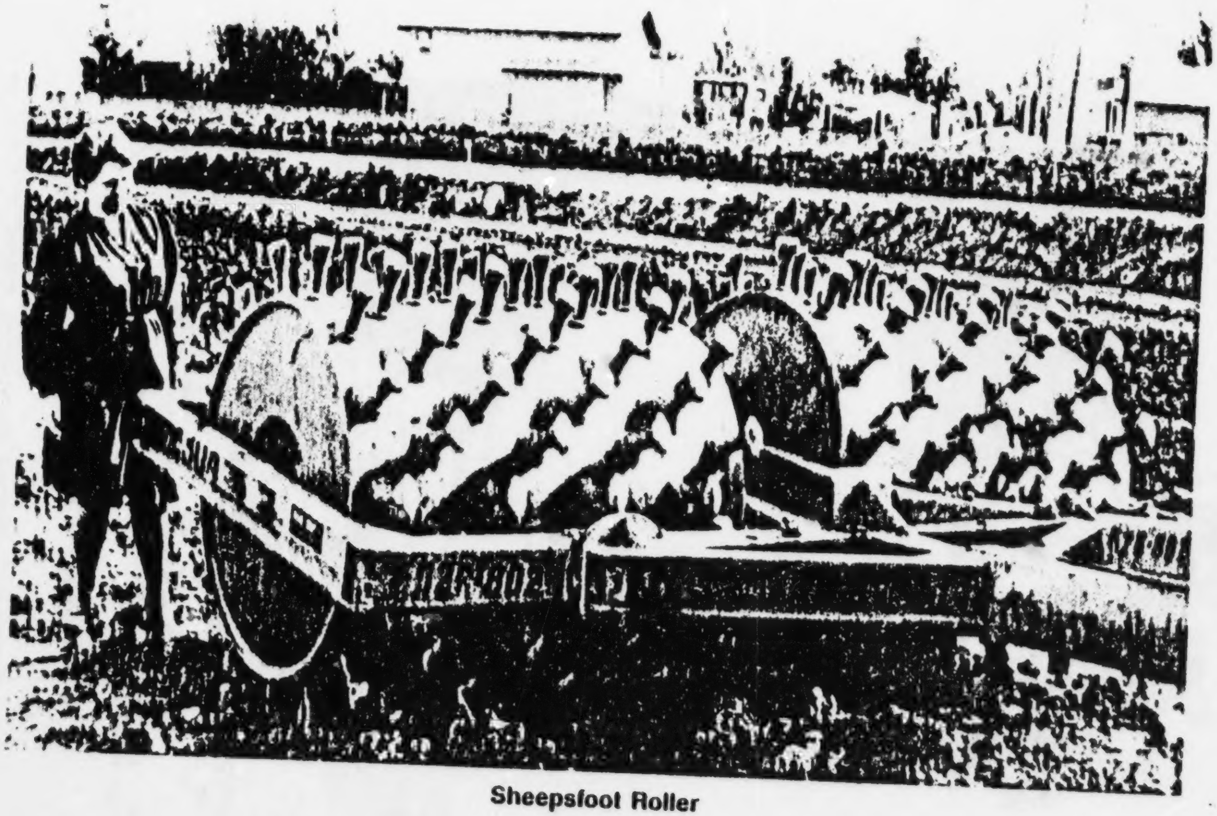


Sheepsfoot Roller

Figure 5-12. Various compacting rollers (continued).

radii. If sidewalls are steep, some equipment may not be able to negotiate the slope. Sidewall slopes of 2.8 to 1 or less have been recommended for sheepsfoot rollers (Boutwell and Donald, 1982). Figure 5-13 shows the compaction of a 2-to-1 slope with a sheepsfoot roller. In this case it was necessary to both push and tow the roller to negotiate the slope. If sidewalls are compacted in horizontal lifts, lift width may restrict the size of the compaction equipment.

The number of passes necessary to achieve the specified compactive effort depends upon the size, weight, and configuration of the equipment. In general, small equipment requires more passes than large equipment. Compactive effort can be estimated from the towing force required to tow the equipment per unit distance multiplied by the number of equipment passes. It is usually expressed on a per unit volume of fill basis (e.g., foot pounds per cubic foot) (Selig, 1982; Johnson and Sallberg, 1960) as follows:

$$\frac{(F)\ (N)\ (L)}{(W)\ (T)\ (L)} = \text{compactive effort per unit volume of fill (ft lb/ft}^3\text{)},$$

where:

F = draw bar pull (lb)

N = number of passes

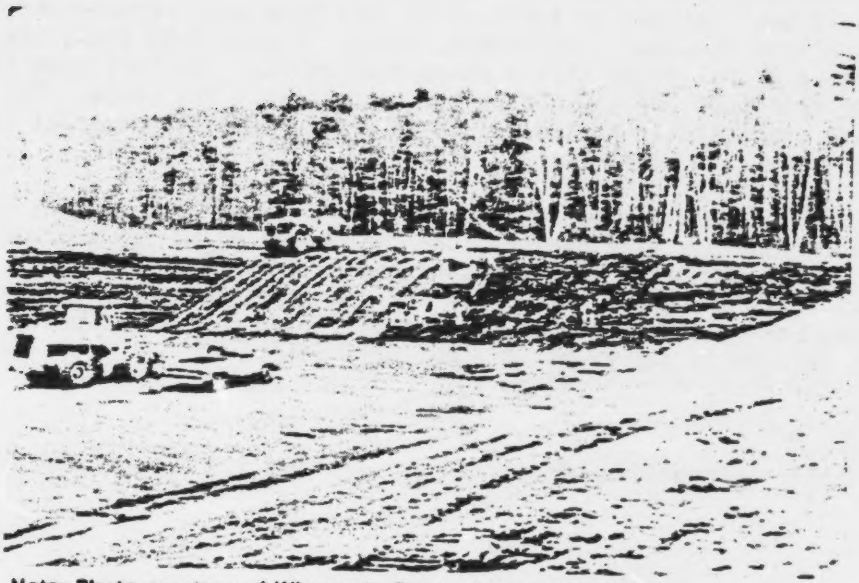
L = length of each pass (ft)

W = roller width (ft)

T = lift thickness (ft).

For each soil/moisture content/equipment combination, a different numbers of passes are required to achieve a specified permeability and density. Thus, it is extremely important to determine the compactive effort necessary to achieve the design permeability with each type of compaction equipment to be used in a test fill prior to construction (Section 5.3.3.1.1). Mitchell et al. (1965) demonstrated that, with some clays, increasing compactive effort can decrease permeability hundredfold without changing density or moisture content by additional shearing that breaks down soil structure. Compaction should be controlled in the field by measuring density, moisture, and compactive effort. The specified values are based on the relationship, previously derived in the laboratory and confirmed in the field test fill, among these variables and the permeability for the specific soil and the specific compaction equipment to be used.

Moisture content, density, and compactive effort measurements are necessary for controlling compaction to ensure that the specified permeability is achieved in the field. Visual observations of construction operations are also critical to compaction quality control. A full discussion of compaction quality control is found in Section 5.3.



Note: Photo courtesy of Wisconsin Dept. of Natural Resources

Figure 5-13. Compaction on a 2(H) to 1(V) slope
with a towed sheepsfoot roller.

5.2.2.2 Admixed Bentonite Liners--

In areas where suitable soils for clay liners are not available or cannot be economically delivered to the construction site, bentonite additives may be blended into the unsuitable native soils to enable them to be compacted to the required permeability. Selection of the type and the proper percentage of bentonite additive is described in Section 5.1.2. The difference between installation of natural soil liners and bentonite admixtures mainly lies in the liner emplacement methods. When bentonite is stored onsite, it is critical to keep it covered and protected from precipitation as it cannot be worked in the wet state.

5.2.2.2.1 Bentonite Mixing and Spreading--When bentonite soil additives are used, they must be thoroughly and uniformly mixed with the native soil. This is most easily done when the native soil is relatively dry. Mixing can be accomplished in place with the additive applied evenly over the site and then mixed into the native soil, or mixing can be done in a central plant where the additives and soil are blended in a mixing device and the final mixture is spread over the site.

Central plant mixing is the preferred method of several design engineers interviewed and in a study by Lundgren (1981) has been shown to be more effective than tilling in place. In this method, the bentonite and the soil are mixed in a pugmill, cement mixer, or other device where moisture is added during the mixing process. Moisture content, particle size, and bentonite content must be monitored and controlled during this mixing process. Central plant mixing is illustrated in Figure 5-14. Laboratory tests of central plant mixing of bentonite and soil yielded the following results (Lundgren, 1981):

- More than 10 minutes mixing time is preferable.
- The soil may be dry, naturally conditioned (drained), or saturated when the bentonite is added.
- Water should be introduced into the mixer after the bentonite and after a couple of minutes of homogenizing.
- All investigated bentonites (five types) were homogenized to the same degree.

Following central plant mixing, spreading may be accomplished as with natural soil liners (using trucks or pans and graders or dozers) or with a continuous asphalt paving machine (Geo-Con, 1984).

Currently, central plant mixers are commercially available that are specifically designed for bentonite/soil admixtures. These mixers are capable of producing 1,000 yd^3/day of admixed material. Computer controls for these devices are capable of achieving an accuracy of ±0.5 percent moisture in the admixture throughout the project (Geo-Con, 1984).

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Note: Photo courtesy of Geo-Con, Inc., Pittsburgh, PA

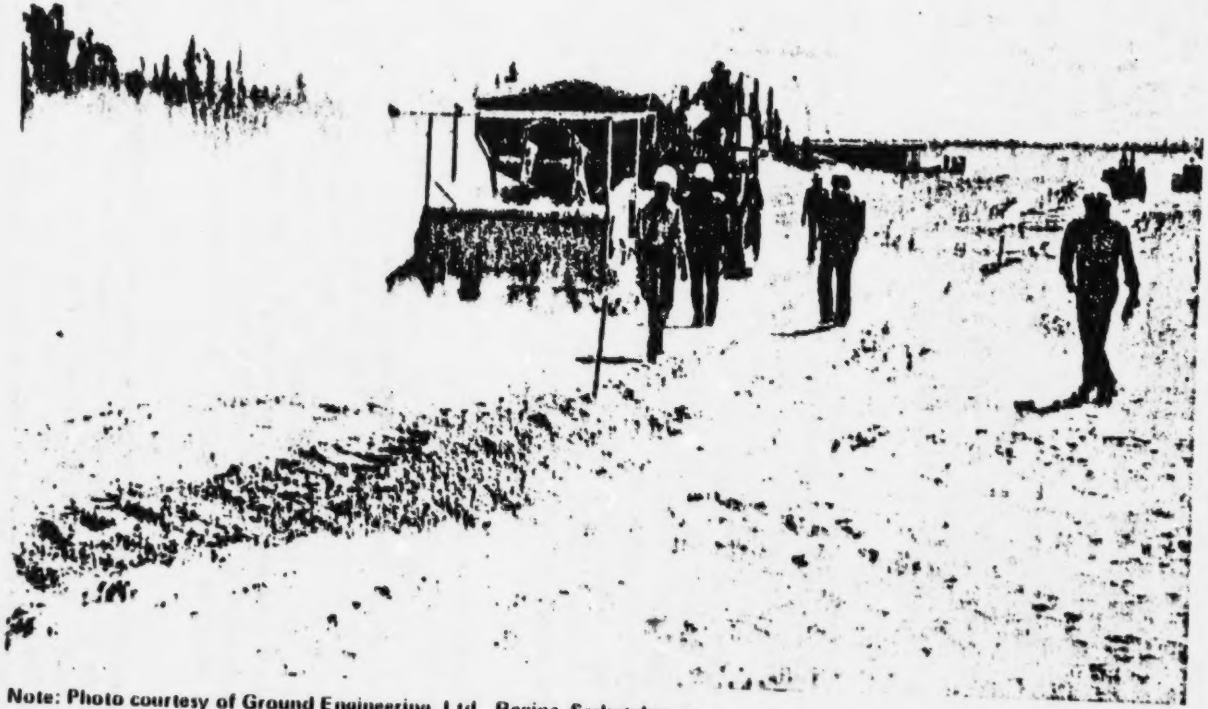
Figure 5-14. Central plant mixing of bentonite and soil.

In-place spreading and mixing is a commonly practiced construction method for bentonite/soil liners. Locally available liner material is first spread uniformly over the prepared foundation. The bentonite is then spread uniformly over the native liner material. If the side slopes are steep (2.5 to 1 or greater) or the liner is small, bags of bentonite can be placed on the site in a predetermined pattern and then the bentonite is manually raked over the liner material. Bag placement must be determined carefully so that the specified quantity of bentonite per cubic foot of liner is maintained uniformly throughout the fill. This is frequently accomplished by placing the bags of bentonite in a grid pattern over the facility site. Close visual scrutiny is necessary during manual mixing to ensure that the spreading is adequate. Alternatively, for larger sites with sidewall slopes of 2.5 to 1 or less, mechanical or pneumatic spreaders can be used. A belt-feed cement spreader has been found to be particularly suitable for bentonite. This spreader, pictured in Figure 5-15, requires only two men and provides uniform spreading rates at up to 25 ton/hr (Kozicki and Heenan, 1983). The spreader can be operated with feed from dump trucks or, when modified with a cyclone, fed pneumatically in bulk from tank trucks (Figure 5-16).

Following in-place spreading, the bentonite must be mixed into the soil along with enough water to bring the mixture to the proper moisture content. This step activates (swell) and disperses the bentonite. Mixing can be accomplished with disk harrows, rototillers, tined rotovators, or a high-speed pulvi-mixer (soil stabilizer). Disks (Figure 5-17) and tillers should make several passes in a crisscross pattern to help break up clods and ensure more complete mixing (Ghassemi et al., 1983). Water may be added from sprinklers attached to the mixer or by sprinkling between passes. Rototillers have been demonstrated to be more efficient at mixing than disk harrows (Ghassemi et al., 1983), and larger wheeled rototillers are more effective than the wheelless types (Lundgren, 1981). Six or eight passes are generally required for adequate mixing.

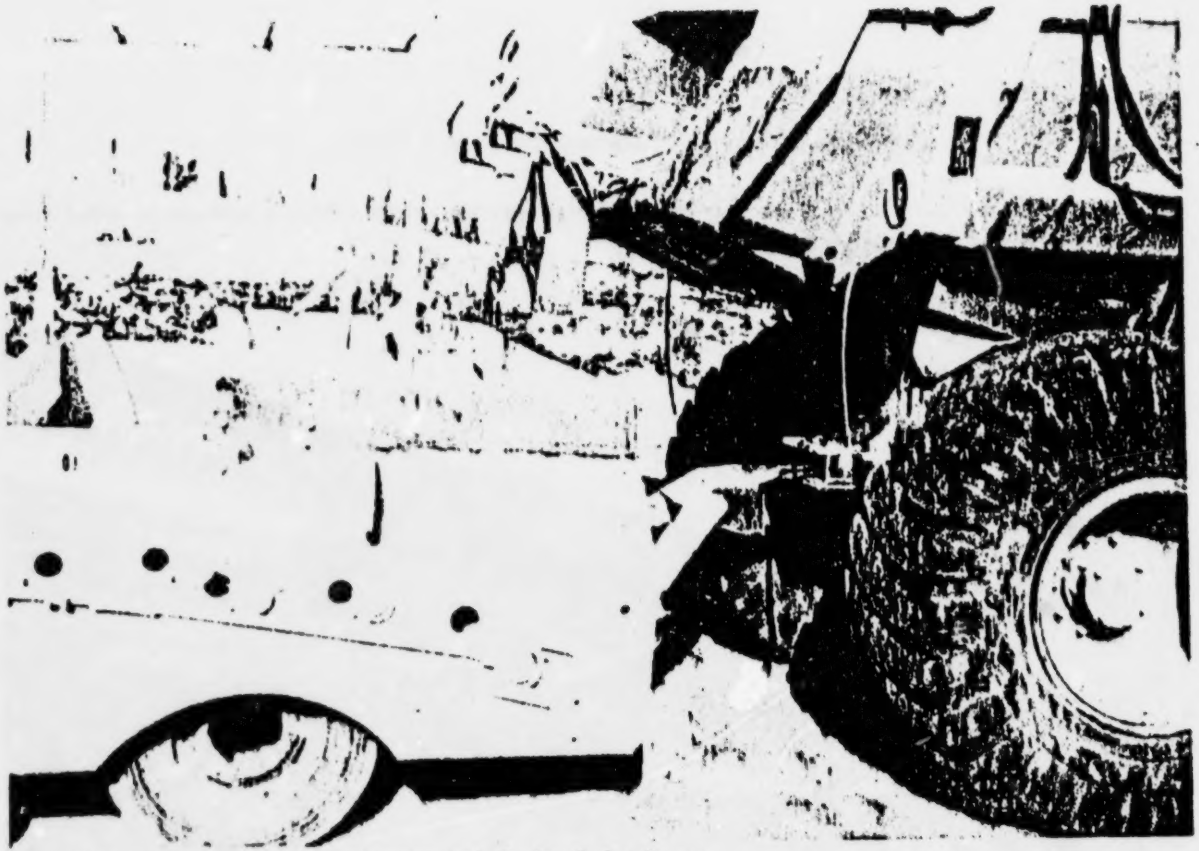
At several Canadian installations, a high-speed pulvi-mixer (soil stabilizer) achieved very good mixing to a depth of 200 mm in the first pass and to a depth of 300 to 350 mm on subsequent passes (Figure 5-18). Water can be added after the first dry-mix pass (Kozicki and Heenan, 1983) or, as illustrated (Figure 5-18), it can be added during mixing. Two passes are generally sufficient to mix the bentonite thoroughly and reduce all clods to less than 1 inch. Although this equipment is considerably more expensive than conventionally used mixing devices, the reduced number of passes and ensurance of better mixing make it cost competitive (Kozicki, 1983, Ground Engineering Ltd., Regina, Saskatchewan, Canada, personal communication).

In-place spreading and mixing are generally recommended only for single-lift (4- to 6-inch) liners; for thicker liners, the central plant method is preferred. One contractor with admixed liner installation experience has stated that because of the difficulty of conducting stringent quality assurance/quality control for in-place spreading mixing, central plant mixing is the preferred method for hazardous waste containment facility liners (Ryan, 1984).



Note: Photo courtesy of Ground Engineering, Ltd., Regina, Saskatchewan

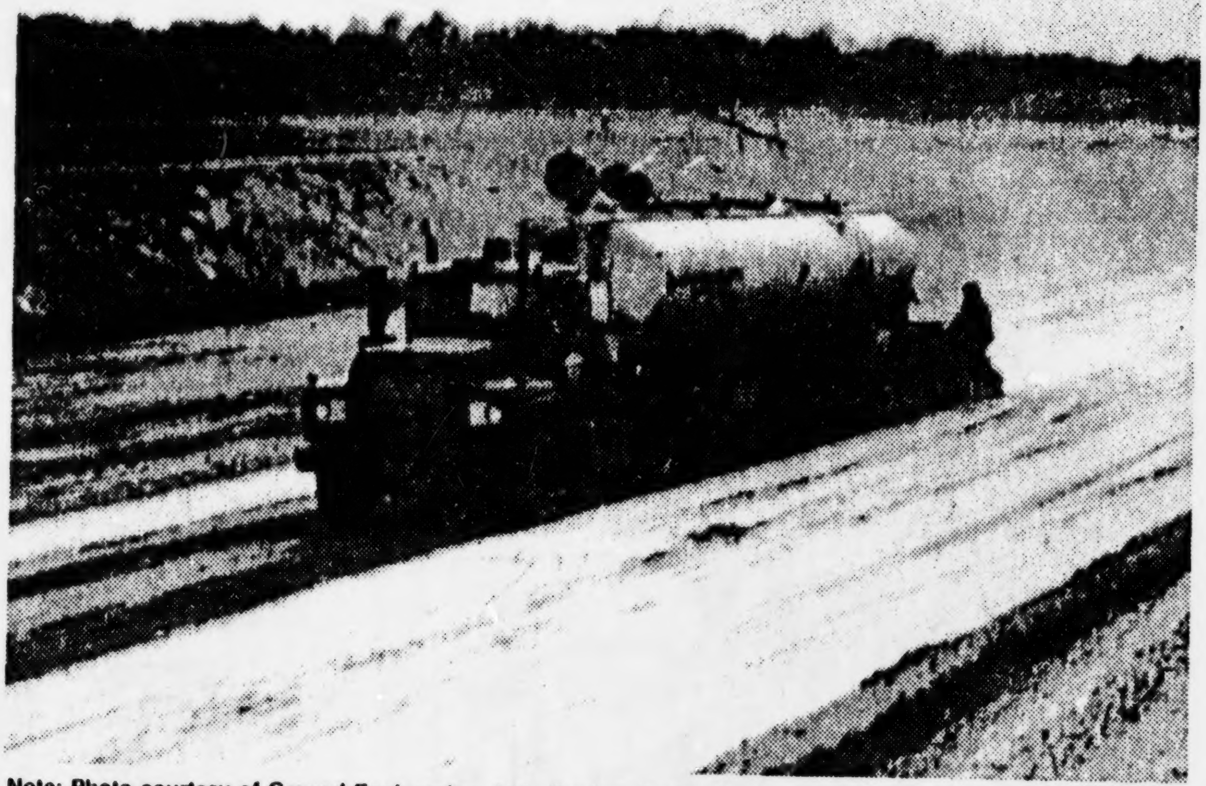
Figure 5-15. Truck-loaded bentonite spreader.



Note: Photo courtesy of Ground Engineering, Ltd., Regina, Saskatchewan

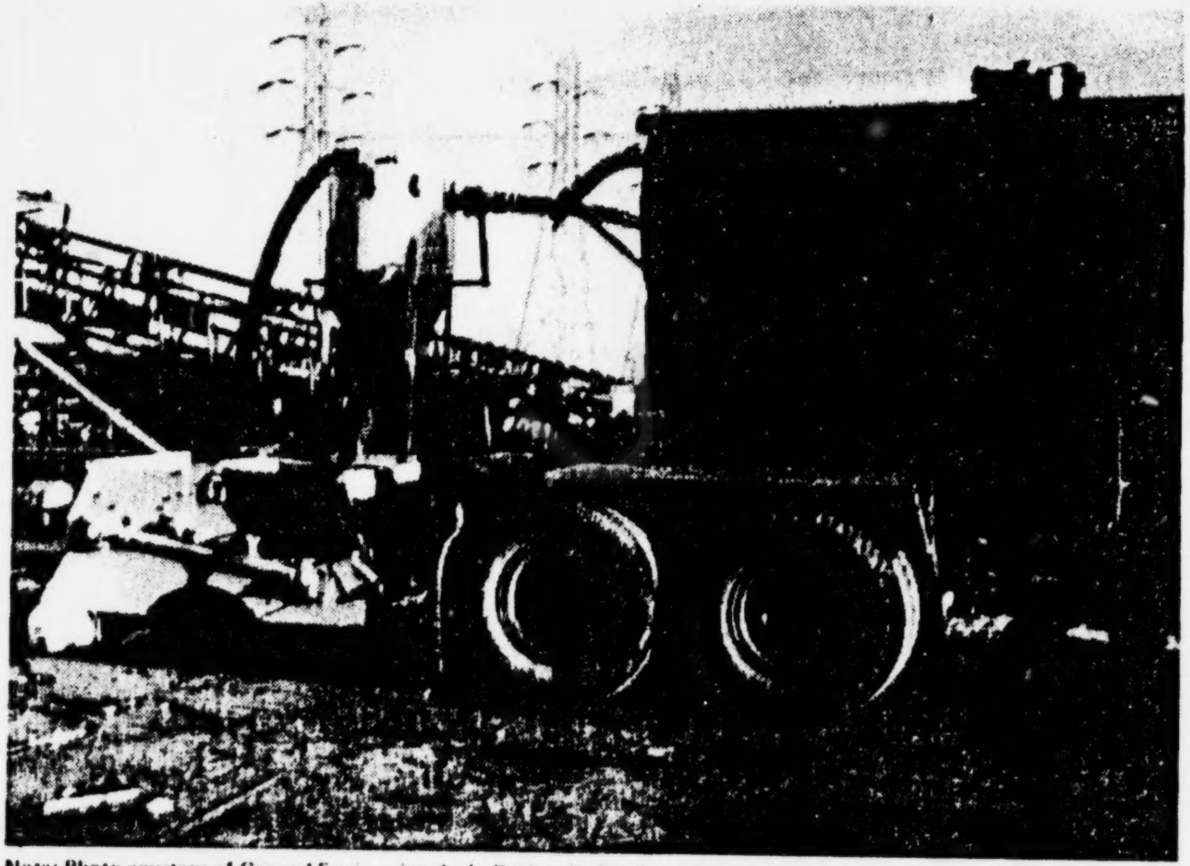
Figure 5-15. Truck-loaded bentonite spreader (continued).

Reproduced from best available copy.



Note: Photo courtesy of Ground Engineering, Ltd., Regina, Saskatchewan

Figure 5-16. Pneumatically fed bentonite spreader.



Note: Photo courtesy of Ground Engineering, Ltd., Regina, Saskatchewan

Figure 5-16. Pneumatically fed bentonite spreader (continued).

Reproduced from best available copy.

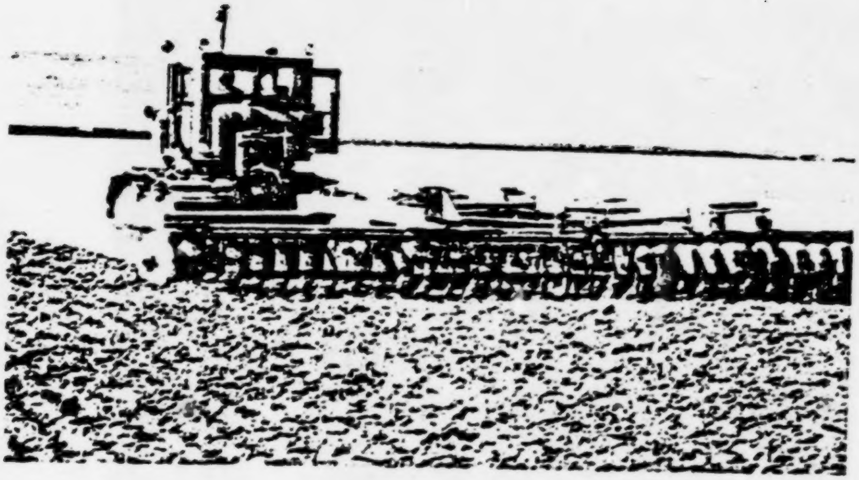
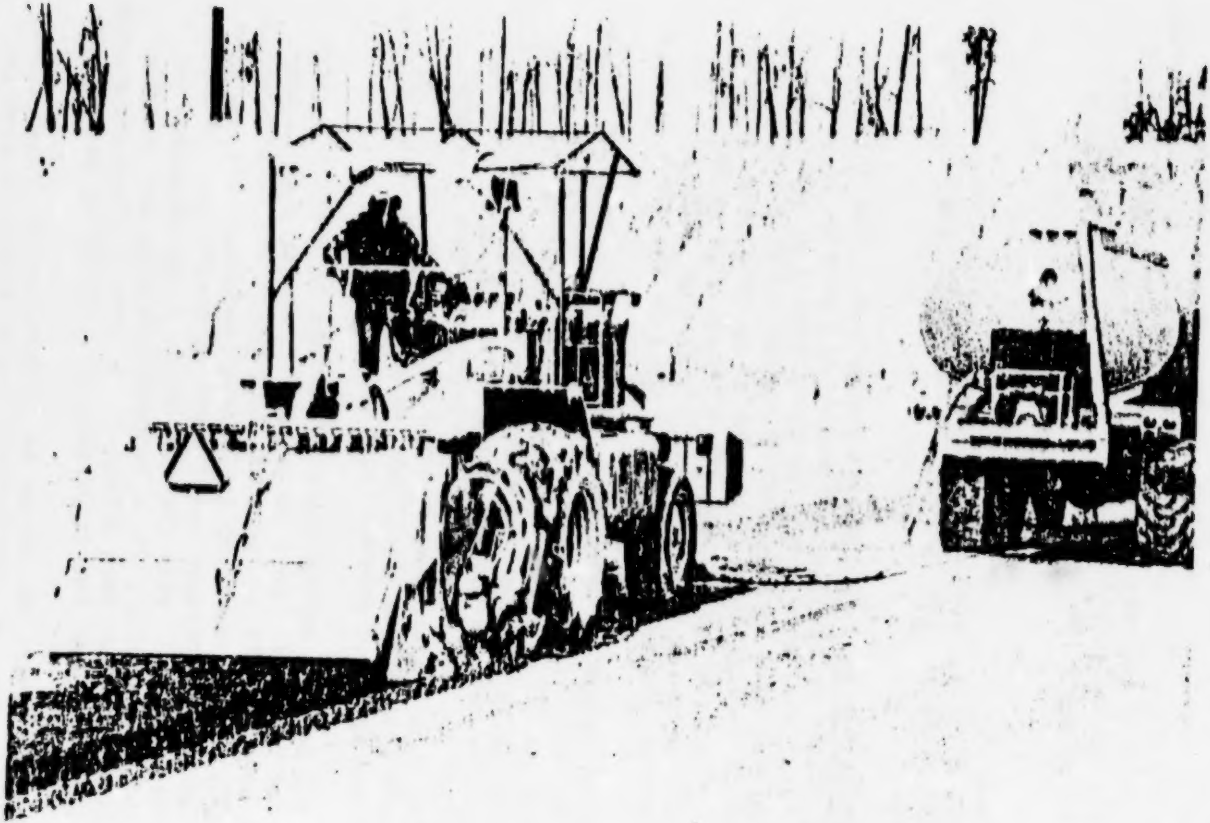


Figure 5-17. Blending bentonite with soil using a disk harrow.



Note: Photo courtesy of Ground Engineering, Ltd., Regina, Saskatchewan

Figure 5-18. Soil stabilizer mixing bentonite in place.

Reproduced from best available copy.

5.2.2.2.2 Compaction--Following spreading and mixing, the bentonite admixture is compacted. Vibratory smooth-wheeled rollers or vibratory-plate compactors are preferred for this operation, for two reasons. First, because admixed liners are often thin (4 to 6 inches), tamping-foot or sheepsfoot rollers can penetrate the liner. However, this may not be an issue for hazardous waste facility liners because most regulations require liners 2 feet thick or greater. The second reason stated for this preference is that the native soils used at these sites often have a high sand content and are most effectively compacted with smooth vibratory rollers (Kozicki, Ground Engineering, Ltd., Regina, Saskatchewan, Canada, personal communication, 1984).

5.2.2.3 Climatic Effects--

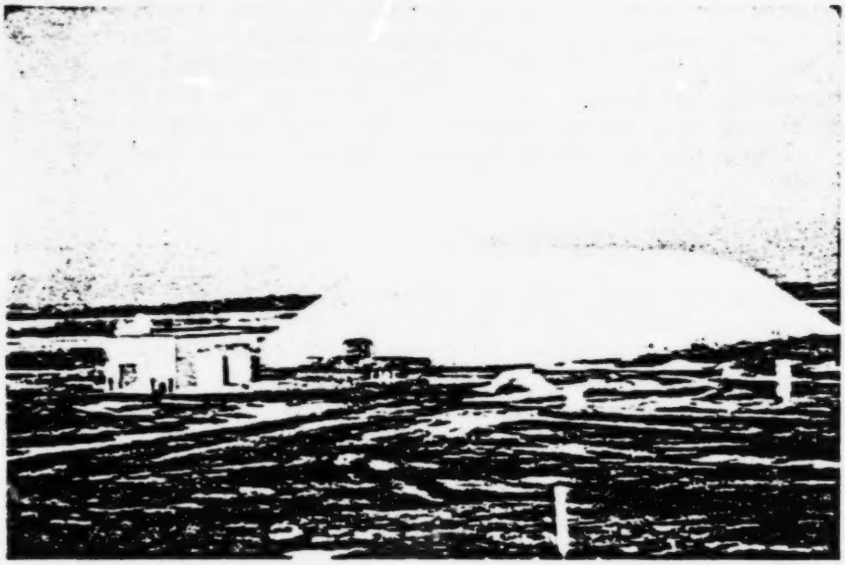
The following section is a discussion of climatic influences on clay liner construction activities and the measures to avoid problems resulting from climatic stresses.

5.2.2.3.1 Precipitation and Desiccation--Precipitation can interfere with construction operations by eroding or flooding the site or by over-moistening the liner material. Provisions for protecting a borrow pile from erosion or overwetting have already been discussed. For liner installation, when construction is interrupted at night or by rain, the compacted lift is usually seal-rolled (rolled smooth with a smooth drum or wheeled roller). If the site is properly graded, this ensures that water will run to the lowest point of the site and not puddle or pond on the liner surface.

Two hazardous waste management companies have used or suggested inflatable domes over secure landfills for protection from the elements during construction and operation (Figure 5-19). These domes enable construction activities to proceed despite inclement weather. In addition, for facilities with continuous operation where wastes are emplaced in the landfill while other parts of the liner are still being constructed, the dome prevents rainwater from falling on the site, thus eliminating the need to collect and treat leachate generated during operation. Upon closure of the facility, the dome is deflated and moved to another location.

Plastic covers may be used during inactive periods to prevent drying or wetting of the liner material. Soil covers are sometimes used to prevent desiccation and erosion. It is important to protect the liner against desiccation, especially if high-swelling soils have been used, because desiccation cracks can seriously increase the liner's permeability. In certain areas, 6- to 8-inch dessication cracks can develop in 1 day (Ghassemi et al., 1983). If desiccation cracks occur, it is necessary to disk and recompact the portion of the liner that has been affected. A liner failure documented in Chapter 7 (Site I) illustrates the importance of controlling desiccation.

5.2.2.3.2 Freezing--Liners should not be constructed of frozen soils. From the standpoint of compaction, frozen soils are difficult to work and the compactive effort needed to achieve a specified density and permeability increases with decreasing temperature, often to the point that the required permeability and density cannot be achieved. Thus, in colder climates, liners cannot be properly constructed during the winter months.



Note: Photo courtesy of Waste Management, Inc., Oakbrook, IL

Figure 5-19. Inflatable dome over a hazardous waste landfill.

Freezing of a liner can cause surface cracking and degradation of the liner soil fabric, resulting in increased permeability (Mercurio, Ebasco Services, Inc., New York, New York, personal communications, 1984). At the end of a construction season, as well as at night in the fall and spring when temperatures drop below freezing, the liner should be protected from freezing. Liners can be protected from freezing with a blanket of soil or organic mulch.

5.2.3 Postinstallation Activities

Upon completion, the liner is rolled smooth to seal the surface so that precipitation and/or leachate can run freely to the leachate collection sump. The completed liner is surveyed to ensure that thickness, slope, and surface topography are as required by the design specifications. Seals around objects penetrating the liner (e.g., antiseep collars around leak detection system pipes) should be checked for integrity. The liner can be covered with plastic or a soil cover to prevent desiccation if any time will pass before the next liner system component (e.g., FML or leachate collection system) is installed or before the liner is covered with waste. This is especially important for bentonite liners or for liners composed of highly swelling native soils where desiccation cracking can occur very quickly.

For surface impoundments, rip-rap can be placed on the upper sidewall slopes to protect them from wave erosion. Experience in dam construction has shown that in most cases rip-rap is best placed by dumping. A U.S. Army Corps of Engineers' survey cited by Small (1981) found that dumped rip-rap failed only 5 percent of the time, whereas the failure rate for hand-placed rip-rap was 30 percent. The failure rate for concrete pavement used as a substitute for rip-rap was 36 percent. Proper sizing of dumped rip-rap is critical to its performance. Rip-rap slopes also must be maintained to provide reliable liner protection. Erosion-resistant structures for channeling or loading liquid wastes into surface impoundments also may be constructed after liner installation.

5.3 QUALITY ASSURANCE AND QUALITY CONTROL

A recent survey of hazardous waste surface impoundment technology revealed that rigorous CQA and CQC are necessary to achieve good site performance (Ghassemi et al., 1983). Liner failures at several impoundments were attributed to various factors including "failure to execute proper quality assurance and control." The success of surveyed facilities that have performed very well is attributed to many factors including "the use of competent design, construction and inspection contractors, close scrutiny, of all phases of design, construction, and CQA inspection by the owner/operator, excellent CQA and CQC and recordkeeping during all phases of the project, and good communications between all parties involved in establishing the sites" (Ghassemi et al., 1983).

Potential causes of clay liner failure that can be avoided with careful CQA and CQC include:

- Use of materials that do not meet the design specifications
- Inadequate foundation preparation
- Inclusions of roots and other organic matter, large rocks, pockets of permeable materials, and other foreign objects in the liner material
- Inadequate moisture control both prior to and after compaction
- Inadequate clod size reduction, mixing, and spreading of liner materials
- Emplacement of inadequate amounts of liner materials (especially important with bentonite/soil liners)
- Failure to follow installation procedures specified in the design
- Use of improper or inadequate construction equipment
- Specification or application of inadequate compactive effort
- Failure to tie lifts together properly
- Inadequate control of soil moisture content and density during compaction and poor maintenance after construction.

This section specifically addresses quality assurance and quality control for the construction of clay liners for hazardous waste landfills, waste piles, and surface impoundments. It addresses liners constructed of both recompacted soil and bentonite/soil admixtures and includes QA activities that are necessary to ensure that the liner material is as specified and that installation procedures will result in a liner that will perform as specified in the facility design. This text is a compilation of information on current CQA and CQC practices obtained in the course of this study through interviews with design engineers and through literature reviews. The section on current practices (Section 5.3.4.4) tabulates much of the information obtained during this effort.

Proper installation of all of the components of a hazardous waste storage or disposal unit (i.e., clay liners, synthetic liners, leachate collecting system, dikes, and cover systems) is necessary to ensure the specified performance of the clay liner and the total containment system. As with clay liners, an adequate CQA and CQC program is necessary during installation of these components to ensure that they will perform as specified; however, a discussion of CQA and CQC for these components is beyond the scope of this document.

5.3.1 Key Terms

The following concepts and terms are used throughout this section.

- Construction Quality Management--The process whereby scientific and engineering principles and practices are used to ensure that a land-based hazardous waste facility is constructed in conformance with its design. The emphasis on construction quality management must begin as early as possible in the design of the facility, must be stressed throughout the actual construction of the facility, and ceases upon closure of the facility. Managing construction quality is the responsibility of the permit applicant, the construction contractor, and the design engineering firm. It consists of two components, CQC and CQA.

- CQA--A planned series of overview activities, the purpose of which is to provide assurance that CQC is being implemented effectively. The system involves a continuing evaluation of the adequacy and effectiveness of CQC, the inspections performed, the data collected, and the interpretation of the data made in response to the CQC activities. For hazardous waste land disposal facilities, this series of overview activities may involve verifications, audits, and evaluations of the factors that affect the installation, inspection, and performance of a hazardous waste land disposal facility to ensure that the facility meets the design specifications.

- CQC--Planned and unplanned inspection activities, the purpose of which is to help provide the level of construction quality that will result in a facility that meets the design specifications. The overall system involves integrating several related factors including: (1) proper selection of specified materials, (2) installation to meet the full intent of the specification, and (3) inspection to determine whether the resulting product is according to the specification.

- CQA Plan--The CQA plan that is discussed in this chapter is a written approach that may be followed to attain and maintain consistent high quality in the construction of hazardous waste storage and disposal facilities. The purpose of a CQA plan is to ensure that a completed facility meets or exceeds all design criteria. The CQA plan is tailored to the specific facility to be constructed and documents the permit applicant's commitment to CQA.

  The CQA plan is prepared as part of the facility design activities. It is usually prepared by the design engineers but in some cases may be prepared by an independent third party responsible for CQA for the facility. The CQA plan indicates what tests and observations will be made during construction to ensure that design criteria are met, as well as test frequency and test spacing requirements for CQC. Acceptance/rejection criteria for specific tests are specified in the plan and reflect the precision of the specific test methods used and the specified values, design tolerances, and expected field variability of the tested parameters. Also specified in the CQA plan are corrective actions to be implemented if some part of the work is substandard and consequently rejected.

While the overall content of the CQA plan will depend on the site-specific nature of the proposed hazardous waste land disposal facility, as a minimum, several specific elements should be included in the plan. These elements are summarized briefly below.

- Responsibility and Authority--The responsibility and authority of all organizations involved in permitting, designing, and constructing the hazardous waste land disposal facility should be discussed fully in the CQA plan.

- CQA and CQC Personnel Qualifications--The CQA officer, CQC inspector(s), and all other CQA and CQC personnel should possess the training and experience necessary to fulfill their identified responsibilities, and their qualifications should be presented in the CQA plan.

- CQC Activities (Observations and Tests)--The specific observations and tests that will be used to control or monitor the installation of the hazardous waste land disposal facility components should be summarized in the CQA plan.

- Construction Quality Evaluation--The sampling activities, sample size, sample location, frequency of testing, acceptance and rejection criteria, and plans for implementing corrective measures, all as contained in the project specifications, should be presented in the CQA plan.

- Documentation--The QCA plan should include a discussion of the reporting requirements for the project. This should include such items as daily summary reports, observation and testing data sheets, problem identification and corrective measures reports, block evaluation reports, design acceptance reports, and final documentation. Provisions for the final storage of all records should also be discussed in the CQA plan.

The following is a more detailed list of important items to be included in a CQA plan to address the above elements properly. These items have been largely adapted from the proven and effective "Contractor Quality Control Plan" of ER 1180-1-6 (U.S. Department of the Army, 1978). They are presented here only as an example and should not be considered an obligatory or complete or exclusive outline. These items are:

- A planned QA organization
- An education plan to ensure that the workmen, construction management, and inspectors are aware of the various quality requirements of the project and the reasoning behind the requirements
- Proposed methods for performing CQC inspections, both for construction process control and for acceptance sampling and testing (quality evaluation); this includes inspections of subcontractors' work
- Delineation of areas of authority and responsibility for each individual in the CQC/CQA group
- Name and qualifications of each individual assigned a CQA or CQC function; method of establishing and verifying personnel qualifications to perform specific tests and/or tasks
- Discussion of how CQC inspections will be performed:
    - by inspectors employed by the permit holder designated by name, their qualifications, and the specific tests and observations to be made
    - by subcontract inspectors designated by name, their qualifications, and the tests and observations to be made subject to approval of the regulatory agency
- The test and/or observation method to be used for each specification quality requirement; whether it is a standard or alternative method; the reference standard or literature reference for each; the method for establishing size of the unit of construction for acceptance testing; the method for establishing sample sizes and sampling locations; acceptance/rejection criteria for each test; and the method for dealing with outlying observations
- Location, availability, applicability, and calibration of test facilities and equipment
- Procedures for advance notice and coordination of special inspections when and where required
- Procedures for reviewing all drawings, samples, certificates, or other documents for compliance with permit requirements and for certification of their acceptability to the regulatory agency; qualifications required of the individual performing the reviews

- Procedures for reviewing inspection test results and observation records; qualifications required of individuals performing the reviews

- Procedures for observing and testing fresh exposures of the site media (soil profile) and for comparing the results with evaluations made during site characterization studies; qualifications required of individuals performing the observations and tests

- Action to be taken by inspectors and reviewers (re: 10, 11, and 12 above) when deficiencies are identified and/or reported, including who is to be notified and in what manner

- Reporting procedures, providing for submittal and/or storage of all test and observation reports, at specified intervals, and reporting of all actions taken under item 13 above; report formats to be used

- Definition of responsibility and authority of all persons and groups involved in the CQA and CQC effort; definition of who has the authority to reject either materials or workmanship and to require removal and/or replacement

- A copy of a letter of direction to the permit holder's inspection representative responsible for CQA, setting forth duties and responsibilities and signed by a responsible official of the permit holder's organization.

5.3.2 Personnel

The individuals or groups of individuals who may be involved in the preparation, approval, and use of a CQA plan include permit applicants (e.g., site owners/operators), permitting agencies, design engineers, construction contractors, CQA officers, and CQC inspectors. The responsibilities and authorities of each of these groups must be well defined and understood by all prior to facility design and construction.

Successful CQA and CQC requires clarity in written and oral communications among all the parties, especially in defining key terms and in delineating areas and lines of authority and responsibility in scope of work and other contractual arrangements. For example, "inspector," "inspection," "certification," and "verification" are terms that are used ubiquitously and frequently without regard to precise definition. The definitions of these terms should be understood and accepted by all parties involved in the clay liner project and clearly documented in a CQA plan.

5.3.2.1 Responsibility and Authority--

The overall responsibility of the CQA and CQC personnel is to execute activities specified under the CQA plan. As a minimum, CQA personnel includes a CQA officer and a CQC inspector. Specific responsibilities and authority of each of these persons are defined in the CQA plan and associated contractual arrangements with the owner. For the CQA officer, specific responsibilities and areas of authority may include:

- Reviewing and fully understanding all aspects of the specified landfill design and proposed construction techniques
- Serving as the owner's or design engineer's liaison with the contractor in interpreting and clarifying contract documents
- Providing CQA reports to the owner on the results of inspections and testing
- Advising the owner or design engineer of work that the CQA officer believes should be corrected, rejected, or uncovered for inspection and of work that may require special testing, inspection, or approval
- Reviewing inspection and test results and rejecting defective work when authorized to do so by the owner or design engineer
- Directing the CQC inspector in performing site inspections and testing
- Stopping construction site activities in cases where deviations from design plans and specifications are detected and implementing corrective actions.

For the CQC inspector, specific responsibilities may include:

- Conducting onsite observations and tests of the work in progress to assess compliance by the contractor with the plans, specifications, and construction-related contractual provisions for the project
- Reporting to the CQA officer results of all inspections including work that does not meet the specifications or fails to meet contract requirements
- Monitoring reviews and tests conducted by the contractor as required by the specifications and contract
- Verifying that tests, equipment, and system startups are conducted by qualified personnel and proceed according to standardized procedures defined by contract documents.

5.3.2.2 Qualifications of CQA Personnel--

Inspectors and CQA officers should be trained in the proper use of all test methods and equipment. They should have the ability to calibrate equipment, administer the required tests, record and interpret data, and make pertinent observations. The training that is required to obtain these skills may come from the classroom or through field experience. However, emphasis is generally placed on first-hand ~~(field)~~ testing experience.

In addition to the above requirements, CQA officers should be registered professionals who thoroughly understand the theory and application of all physical and observational tests, the overall site design, the proper use of the various types of construction equipment, and project management and who have sufficient practical experience in landfill construction.

5.3.3 Observations and Tests

This section describes the observations and tests that should be specified in a CQA plan for clay liner construction. The following section is divided according to the CQA and CQC activities that will take place during preconstruction, construction, and postconstruction periods of clay liner installation activities. All ASTM test methods referenced in this chapter may be found in ASTM (1985).

5.3.3.1 Preconstruction--

The first activity under preconstruction CQA is to review the design drawings and construction specifications for the clay liner that is to be installed with emphasis on CQA and CQC. The design drawings and construction specifications need to be clear and understandable from the standpoint of both the onsite CQC inspectors and the contractor. If the design is deemed inadequate or unclear by the CQA officer, it should be returned to the design engineer for clarification and/or modification.

Prior to construction, the CQA officer must also assess the capabilities of the construction contractor's personnel so that he can determine the type and amount of training, instruction, and supervision that will be needed during construction operations. The contractor's prior performance in general earthwork activities, experience in construction of hazardous waste facilities, and experience in working the specific type of soil and equipment to be used in constructing the facility in question need to be addressed in this assessment.

A preconstruction training plan should be included in the CQA plan. as stated by the U.S. Department of the Army's Construction Control Manual (1977):

> Preconstruction instructions and training should be given to field inspection personnel to acquaint them with design concepts and to provide them with a clear understanding of expected conditions, methods of construction, and the scope of plans and specifications. This may be done by training sessions, preferably with design personnel present, using a manual of written instructions prepared especially for field personnel, to discuss engineering considerations involved and to explain control procedures and required results.

If significant quantities of the soil liner material are to be stored at the construction site, the CQA officer should make provisions to ensure that storage facilities are adequate to prevent alteration of the liner material. For natural clay soils, this could include seal-rolling, grading, or covering the storage pile to encourage runoff and to reduce erosion. If bentonite/soil admixtures are to be used as liner material, it is critical that bentonite material stored onsite is sheltered from precipitation because this material cannot be worked except when in a dry state.

The CQA plan must provide assurance that any liner material brought onto the site is as specified. Soil material screening begins as a preconstruction activity and continues during construction as long as material is being brought onsite or excavated from the site. This activity can be accomplished in several ways, depending on the source of the liner material and site conditions. If the liner material is obtained onsite, the inspection can be accomplished as it is placed in the borrow pile for storage. If the excavated soils are heterogeneous, it may be necessary to segregate the soil material as it is excavated, with suitable soil placed in a borrow pile for future use and soil that does not meet specifications discarded. The CQC inspector observes the segregation operations carefully to ensure that only suitable material is retained for liner construction.

Similarly, if the liner soil is obtained from a nearby borrow area, the soil material may be inspected at the borrow site or as the material arrives at the construction site. Borrow site inspection is more desirable, especially if the soil is heterogeneous, because this will ensure that only suitable material is transported to the site, saving transportation costs. Subsurface characterization of the borrow site may expedite this effort for heterogeneous borrow sources. The U.S. Department of the Navy (1982) recommends that during initial exploration of the borrow pit area, borings or test pits be made on a 200-foot grid. If variable conditions are found during this initial exploration, intermediate borings or test pits should be made. Borrow pit exploration should produce the following information:

- A reasonably accurate subsurface profile down to the anticipated excavation depth
- Engineering properties of each material considered for use
- Approximate volume of each material considered for use
- Water level
- Presence of sands, gypsum, or other undesirable materials
- Extent of organic or contaminated soils, if encountered.

For extremely heterogeneous borrow areas, it may be necessary for the inspector to guide the excavating equipment to avoid substandard soil material.

Inspection of the soil can be largely visual; however, CQC personnel conducting this inspection must be experienced with visual-manual soil classification techniques (ASTM D 2488). Changes in color or texture may indicate a change in soil type or soil moisture content. The soil also may be inspected for roots, stumps, and large rocks. In addition, as a check of visual observations, samples of the soil usually are taken and tested to ensure that the soil's index properties are within the range stated in the specifications; tests of moisture/density characteristics (ASTM D 698) also should be conducted to ensure that these relationships do not change. The number of tests to be conducted depends on site-specific conditions (i.e., soil type and heterogeneity) and the experience of the CQC personnel. Usually a minimum number of tests per cubic yard of material is specified, with additional tests required by the inspector if visual observations suggest a change in soil type.

Soil index properties are simplified tests that provide indirect information about the engineering properties of soils beyond what can be gained from visual observations. Although the correlation between index properties and engineering properties is not perfect, it is generally adequate for CQC purposes. Index property tests commonly used to screen soils are described below.

Atterberg limits include the liquid limit and the plastic limit (ASTM D 4318). These tests are commonly used along with grain size distribution for monitoring changes in soil type. A significant change in Atterberg limits usually reflects a change in important engineering properties, such as the relationship among moisture content, density, compactive effort, and permeability.

Grain size analysis is another important screening test for changes in soil composition. The percentage of clay-size particles and the overall particle size distribution of a soil affects its engineering properties, especially permeability and strength. Rough estimates of grain size may be obtained through manual estimates (ASTM D 2488) and may be sufficient for screening. A 200-mesh sieve may be used to separate coarse (sand and gravel) and fine (silt and clay) particles. More detailed grain size distributions may be obtained by sieving the coarse fraction and by using several settling methods (hydrometer, decantation, or pipette) for the fine fraction (ASTM D 422). Cases where samples of incoming or installed liner materials contain inadequate quantities of the necessary soil particle size may be referred to the CQA officer for evaluation. Moisture content and consistency tests are also needed for screening soils. Again, it is important to monitor carefully for soil type changes as long as liner material is being placed. A change in soil type requires the compaction of another test fill as described in the following text.

When bentonite/soil liners are specified, incoming bentonite should be inspected to ensure that its quality is as specified. For all bentonite shipments, certification of compliance with material specifications should be obtained from the manufacturer or supplier. In addition, the quality of the arriving bentonite should be tested frequently for dry fineness, pH, and viscosity and fluid loss of a slurry made from the bentonite. Dry fineness is the percentage passing a 200-mesh sieve. It is necessary to control dry fineness to ensure proper mixing of the bentonite. Slurry viscosity, slurry fluid loss, and pH are standard tests specified by the American Petroleum Institute (API, 1982). These tests are necessary to ensure the quality of the bentonite in terms of its swelling potential. Electric pH meters should be used for pH measurements; pH papers are usually not reliable (Xanthakos, 1979). If bentonite additives are specified, the manufacturer's certificate contained with each shipment should state compliance with the specified characteristics. More information on testing bentonite quality may be found in Xanthakos (1979).

5.3.3.1.1 Test Fill--An important preconstruction CQA activity is to determine the suitability of the equipment and methodology to be used to compact the liner. To accomplish this, most design engineers recommend that a representative test fill of liner material be compacted with the designated equipment to see if the specified density/moisture content/permeability relationships determined in the laboratory can be achieved in the field with the compaction equipment to be used and at the specified lift thickness. Test fill dimensions must be sufficient to accommodate the compaction equipment. Several lifts are usually compacted in the test fill to check the methodology to be used to tie lifts together. The test fill also is used to determine the number of equipment passes (or amount of compactive effort) needed to achieve the specified permeability and to determine the ability of mixing equipment to break up large clods of uncompacted liner soil.

Field permeability tests can be conducted on the compacted test fill material. Field compactive effort is different from the compactive effort applied in the laboratory. Although densities may be the same for different types of compactive efforts, the fabric and the permeability of soil compacted by different methods can differ significantly (Mitchell, 1976). These permeability tests, therefore, are necessary to ensure that the compactive effort that is applied in the field will result in the same or lower permeability than was demonstrated in the laboratory tests of the liner material. Additionally, the test fill is useful in establishing a relationship between field permeability and laboratory permeability measurements on undisturbed samples of compacted liner material, which can vary by as much as 3 orders of magnitude (Day and Daniel, 1985). The long time necessary for field permeability measurements often limits their use during construction operations because of scheduling problems. However, field permeability measurements can be scheduled conveniently during test fill compaction.

In addition to the number of passes necessary to achieve the specified permeability, equipment type, size, and compatibility with the soil type are evaluated and recorded during test fill compaction. Equipment items to be checked and recorded include:

- For sheepsfoot rollers--drum diameter and length, empty and ballasted weight, length and face area of feet, and the yoking arrangement

- For rubber-tired rollers--the tire inflation pressure, spacing of tires, and empty and ballasted wheel loads

- For vibratory rollers--the static weight, imparted dynamic force, operating frequency of vibration, and the drum diameter and length (U.S. Department of the Army, 1977).

This information is necessary to estimate the compactive effort and compactive force applied in the field.

Compacting the test fill is also valuable in that it gives the CQA officer, CQC inspector, and construction personnel some experience with the behavior of the specific soils, equipment, and methodology to be used at the site during compaction. This is a very useful base of knowledge to apply when observing compaction of the actual containment facility liner. The test fill compaction thus serves to calibrate the inspectors' and contractors' observations to the conditions that will be encountered during the compaction of the liner.

The successful use of density and moisture content measurements for CQC of clay liner compaction depends upon the relationship established among density, moisture content, compactive effort, and permeability for the specific soil and for the specific compaction equipment and methodology used during test fill compaction. If a change in soil characteristics or a change of compaction equipment or methodology occurs during liner construction, another test fill should be compacted with the new soil, equipment, and/or methodology because the original relationships may no longer apply. If this is not done, there is no assurance that CQC using density/moisture content/compactive effort measurements will result in a liner with the specified permeability.

5.3.3.2 Construction--

An important CQC activity during clay liner installation is observation of the construction process, including personnel performance, by the inspectors. Observations by an experienced inspector, coupled with a soundly developed and implemented CQC testing plan, will ensure that the liner is installed as specified, that any potential problems are identified in a timely manner, and that proper corrective actions are implemented.

5.3.3.2.1 Foundation Base Preparation--CQC for excavation and construction of foundations is not fully addressed in this document. Standard methods for controlling the quality of foundation preparations and earthen embankments may be found elsewhere (e.g., Spigolon and Kelley, 1984; U.S. Department of Interior, 1974; U.S. Department of the Army, 1977). The foundation base must be adequately prepared before the clay liner is constructed. The natural foundation should provide satisfactory contact with the overlying compacted liner, minimize differential settlements and thereby prevent cracking of the liner, and provide an additional barrier to leachate migration from the facility. To ensure that these goals are met, observations during the construction of foundations should include the following (U.S. Department of the Army, 1977):

- Observations of stripping and excavation to ensure that all soft, organic, and otherwise undesirable materials are removed. Proof-rolling with heavy equipment can be used to detect soft areas likely to cause settlement. Consistency of the foundation soil may be checked with a hand penetrometer, field vane shear test, or similar device.
- Inspection of soil and rock surfaces for adequate filling of rock joints, clay fractures, or depressions and removal and filling of sand seams.
- Inspection of the depth and slope of the excavation to ensure that it meets design requirements.
- Observations to ensure proper placement of any recessed areas for collection or detection pipes and sumps.
- Tests and observations to ensure the quality of compacted fill.

Visual observation of the construction process is the major means of ensuring that the foundation is constructed as designed. Some instrument surveying may be necessary to ensure that facility dimensions, side slopes, and bottom slopes are as specified. Visual-manual soil identification techniques (ASTM D 2488) and index property tests (ASTM D 422; D 4318) may be used to monitor foundation soil composition.

Soil consistency may be checked with a cone penetrometer (ASTM D 3411). This method is widely used to determine the consistency of cohesive soils for classification and is more accurate than the visual-manual method. It is less accurate than the laboratory method (ASTM D 2166), but can be used to give broad consistency classifications for cohesive soils. Shear strength of foundation soils can be checked in the field with a field vane shear device (ASTM D 2573) or a torvane device (Lanz, 1968). The ASTM D 2573 method is standardized and, although it is less accurate, can be correlated with the standard laboratory method for unconfirmed compressive strength (ASTM D 2166). Although all of these field expedient methods give only approximate values, they are usually sufficient for construction. Compaction is controlled as described in Section 3.3.3.2.3.

Further information on quality control of foundations may be found in Spigolon and Kelley (1984), U.S. Department of the Interior (1974), and U.S. Department of the Army (1977).

5.3.3.2.2 Liner Lift Placement--Liner lift placement includes the operation of spreading the liner material(s) over the floor of the facility; breaking and homogenizing clods of soil; and, for blended clay liners and admixed liners, uniformly blending the mixed materials.

During placement of soil materials, the soil is spread uniformly as specified. The loose lift thickness of the soil should be measured systematically over the entire site, with a marked staff or shovel blade, and survey levels should be made every few lifts for verification and documentation of liner thickness. Following spreading, the liner material is disked or tilled to break up large soil aggregates and to homogenize the material. All large clods of liner material should be reduced in size as much as possible to facilitate moisture penetration and to ensure uniform compaction through the lift. Opinions differ on the acceptable maximum clod size; in a series of interviews with design engineers, recommendations ranged from 1 inch to no greater than the lift thickness (see Table 5-8 in Section 5.3.3.4). Close observation by the onsite CQC inspector is critical to ensure that this is properly accomplished.

If bentonite additives are to be admixed with the natural soil, the proper percentage of the additive must be controlled. For spreading and in-place mixing, this is accomplished by visual observations of the additive as it is spread over the site by hand or by a suitable spreading device. In addition, a spreader can be roughly calibrated by using it to spread the additive over a plastic sheet or pan of given dimensions and then weighing the collected material. Visual observations also should be made to ensure that additives are uniformly distributed in each liner lift. To ensure the proper bentonite percentage in admixtures, the methylene blue test (see Appendix A) can be carried out on representative samples of the admixed material. The soil mixture's liquid limit or plasticity index has also been used to ensure proper bentonite percentage.

If bentonite additives are to be blended with the native soil prior to emplacement, proper bentonite percentage can be ensured by using the methylene blue test on samples of the mixed material. CQC personnel also can inspect the mixing operations to ensure that the proper amounts of bentonite and soil are placed into the mixing device and that mixing time and force are sufficient.

5.3.3.2.3 Moisture Control--Moisture content of the liner soil should be measured and controlled both before and after lift placement to ensure that the soil moisture content is as specified and for activating bentonite additives. Nuclear probes (ASTM D317-78) and/or manual moisture content measurements are generally used to control moisture in the field (see Chapter 3). The following description of manual moisture estimation is adapted from Johnson et al. (1983).

> In order for quality control personnel to satisfactorily estimate fill water contents, they must become thoroughly familiar with the fill material prior to the start of fill operations. Preferably, they should spend some time in the field laboratory, performing several compaction tests to become familiar with the differences in appearance and behavior of the various fill materials, to recognize when they are too dry or too wet, as well as when they are at the specified water content. Inspection personnel should also run several Atterberg limits on fine-grained soils so

> they can compare the appearance and feel of the soils when they are at the plastic limit with that at proper water content for compaction.
>
> Trained personnel should then be able to pick up a handful of soil and make a reasonable estimate of the relation of its water content to its optimum water content by feel and appearance (experienced technicians often can estimate deviation of water content from optimum within ±1 percent). Occasional moisture tests should be made to confirm these manual estimates.

A few reference bags of liner material at known moisture contents stored at the site can be very helpful for calibration of manual moisture content estimates.

Many design engineers recommend that oven moisture content measurements by the appropriate test method (ASTM D2216-80) be taken at intervals to ensure that nuclear measurements and manual estimates are accurate. Proper moisture content is achieved and maintained with sprinkling devices (if too dry) or by a combination of mechanical agitation, aeration, and solar drying (if too wet).

If the liner is to be left exposed between the installation of successive lifts or after completion, the QC personnel make sure that it is protected from moisture content change by ensuring that seal-rolling is uniformly performed over the site and that cover material, if used, is properly emplaced. If desiccation does occur, the QC inspector reports this to the QA officer, who notes this and specifies remedial actions, such as disking and recompaction of the affected portion of the liner.

5.3.3.2.4 Compaction--Soil selection based on laboratory compaction and permeability tests is made on samples of the liner material during design of the facility. A relationship among moisture contents, densities, and permeabilities is established based on test results. Design specifications usually require achievement of a minimum percentage of the maximum density (Proctor or modified Proctor) at a specified range of water contents, based on these results. The specified density/water content corresponds to the density/water content at which the specified permeability can be achieved as established by the laboratory and test fill compaction tests. This density/water content is then tested during quality control of clay liner installation.

Additionally, during compaction of each lift, compactive effort and uniformity of compaction are observed and recorded. Compactive effort is estimated by the number of passes of equipment of a known size and weight. It is important to make coverage uniform, especially at fill edges and in equipment turnaround areas and at the top and bottom of slopes (Spigolon and Kelley, 1984). In addition, permeability tests often are conducted periodically on the compacted liner material as a check on the moisture/density/permeability relationship. It is generally agreed that all of the above measurements are necessary to ensure that the specified permeability is being achieved in the field. Density measurement should never be used alone for quality control of clay liner installation.

The relationship among moisture content, density, compactive effort, and permeability is unique for a specific soil or soil mixture and specific type of equipment. Laboratory compaction tests to establish density/moisture curves are determined regularly on field samples of the clay liner to determine changes in optimum water contents. If these tests or field inspection of the incoming liner materials indicate a significant change, laboratory permeability tests and a test fill compaction must be conducted to establish the density, moisture, and permeability relationship for the new soil. Otherwise, attainment of the moisture/density relationship that was specified for the original soil may result in an unacceptable permeability with the new soil. Similarly, if different compaction equipment or methodology is used, another test fill should be compacted with the new equipment because the type of compactive effort applied affects the final permeability.

The design engineers interviewed during this investigation stressed the importance of visual and manual observations by a qualified inspector for liner compaction quality control. Some consider specific tests only a backup documentation to observations by the qualified CQC inspector. All professionals interviewed agreed that observation of the construction process is the primary and most effective approach to CQC. Testing is secondary; beyond the minimum test frequency and spacing, visual observations are used to identify problem areas and to call for more intensive testing to document and delineate any substandard liner areas. Remedial actions (e.g., removal and reconstruction) are then ordered for the substandard areas so delineated. All engineers stressed the importance of having a qualified inspector on the site at all times during construction.

An experienced observer can determine how compaction is proceeding (e.g., moisture content and density) by observing how the equipment moves across the area to be compacted, how the soil contacts or sticks to the compaction equipment, how the soil heaves during compaction, how deep the compaction equipment sinks into the soil, and how the roller walks out of the soil (sheepsfoot rollers only) (Johnson et al., 1983). In addition, it is important to observe uniformity of coverage by compaction equipment, especially at fill edges, in equipment turnaround areas, and at the top and bottom of slopes (Spigolon and Kelley, 1984).

5.3.3.2.5 Specific Tests--Testing and sampling of the liner are necessary to ensure compliance with the design requirements and to document the as-built conditions of the clay liner. Specific tests to ensure that compaction results in the specified liner permeability include field density tests (nuclear, sand-cone, and others), field moisture content measurements, laboratory compaction tests, and both field and laboratory permeability tests. The methods and QC measures for conducting these tests may be found in several documents (AASHTO, 1978; ASTM, 1985; U.S. Department of the Army, 1970; and U.S. Department of the Interior, 1974) and are briefly discussed in Appendix A. The main tools used for controlling the quality of compaction are field density and moisture content measurements, with supplementary laboratory compaction tests to monitor changes in soil material. A laboratory compaction test should be conducted for every 10 to 20 field density/moisture determinations, depending on soil variability (U.S. Department of the Navy, 1982). Presently, nuclear probes are often used to measure field density and moisture content because of ease and quickness of testing (see Chapter 3). However, nuclear devices must be calibrated for each soil that is to be tested. In addition, if nuclear devices are used, other field density and moisture content measurements, such as sand cones and oven drying, should be made periodically to confirm nuclear results. Again, it is necessary to measure density, moisture, and compactive effort in the field to ensure that the required permeability is achieved during clay liner compaction.

In addition to density and moisture measurements and estimates of compactive effort, permeability tests should be made regularly to confirm that the measured moisture/density levels correspond to these required for the specified permeabilities. Shelby tube or block samples may be taken for laboratory permeability tests (ASTM D 1587-74; ASTM, 1983), or field permeability tests may be performed with a double-ring infiltrometer (ASTM D 3385-75; ASTM, 1983) or similar field permeability device (Appendix A). Laboratory permeability tests are easier to conduct, do not consume valuable construction time, and are quicker than field permeability tests. However, they can underestimate field permeability by as much as 3 orders of magnitude (Day and Daniel, 1985). Field permeability tests more accurately represent actual permeabilities, but they can take days to complete, seriously interrupting construction activities. Field permeability tests should be conducted on the test fill prior to construction. Field permeability tests can be accommodated easily during this phase because they will not interrupt construction activities. (For further discussion of field versus laboratory permeability measurements and for a discussion of test methods, see Daniel, 1981; Olson and Daniel, 1981; and Rogowski and Richie, 1984; also see Chapter 3 of this document.)

Several design engineers recommended that moisture/density measurements and Shelby tube samples for laboratory permeability tests be obtained from the lift underlying the lift that has just been compacted. These engineers believe that during compaction of a lift, significant compactive effort is being applied to the underlying lift. A more accurate representation of the degree of compaction across the liner is achieved by testing the underlying lift.

Following Shelby tube sampling or nuclear density measurements, the resulting hole is filled with liner material and hand tamped or is grouted with bentonite. Over excavation of the hole with a shovel to slope the sides of the hole prior to backfilling may further facilitate sealing. Test locations should be staggered from lift to lift so that the testing or sampling holes do not line up. This also gives better test coverage of the liner.

Index property tests (e.g., grain size, clay content, and Atterberg limits) are used to evaluate liner soils prior to emplacement in order to monitor changes in soil type. These test methods are well established and are described in Chapter 3 and in Appendix A.

Minimum test frequency and test spacing should be specified for all tests in the test plan. Test spacing practices and frequency are discussed in Section 5.3.3.4 of this report.

5.3.3.3 Completion Tests--

Upon completion of the liner, CQA personnel should check that it is rolled smooth to seal the surface so that precipitation and/or leachate can run freely to the leachate collection sump. The completed liner should be surveyed to ensure that thickness, slope, and surface topography are as required by the design specifications. Seals around objects penetrating the liner (e.g., leak detection system stand pipes) also should be checked for integrity.

Field permeability tests should be conducted on the completed liner as a final QA check. It appears that field measurement of permeability is preferable to laboratory measurement because it subjects a larger portion of the liner to permeability testing. Recent work has shown that field permeability measurements yield an average hydraulic conductivity close to a liner's actual hydraulic conductivity. Laboratory tests, even on undisturbed samples, gave a hydraulic conductivity 1,000 times less than the actual value, measured by collecting seepage through the liner (Day and Daniel, 1985). Several field permeameters can be set up over the site, or if the site is not too large or is a surface impoundment the facility can be filled with water and seepage from the site can be measured after accounting for evaporation. The latter method, when feasible, is the best way to ensure that the entire site will function according to specifications once it is filled, assuming that no waste/liner compatibility problems occur.

If the completed liner is to be left exposed prior to installation of the overlying facility components, CQC inspectors should ensure that the liner is covered adequately with soil or plastic sheeting to prevent desication and wind erosion or that it is maintained at suitable moisture content by managed water application.

5.3.3.4 Current Sampling Program Design Practices--

Current CQC practice relies on the inspector's observations and judgment for accepting most of the work, with actual sampling and testing of the material serving only to document compliance and to help guide the inspector's judgment. Table 5-8 is a summary of information on current CQC practices obtained during interviews with design engineers active in designing and controlling the construction quality of clay liners. Also included in this table is information from earthwork construction manuals of the U.S. Department of the Interior and the U.S. Department of the Navy. Tables 5-9 and 5-10 list recommendations for construction documentation recently published by the Wisconsin Department of Natural Resources personnel.

TABLE 5-8. CURRENT QA PRACTICES FOR CLAY LINER CONSTRUCTION

Source of information[a]	Minimum frequency of in-place moisture or density tests	Permeability testing	Sample or test hole filling method	Test fill recommended	Maximum clod size allowed	Maximum rock or root size allowed
A	1 per 1,000 yd^3 of fill	Occasional lab test	--	Not usually	½ lift thickness; none greater than lift thickness	--
B	Moisture tests only (used with permeability tests and compactive effort to control compaction)	Triaxial and some field tests	Bentonite	Yes	2 inches	2 inches
C	Frequency per unit volume of fill based on experience	Lab tests	--	--	½ lift thickness	--
D	1 per 2,500 ft^2 of lift	Triaxial--1 per 3 acre feet with minimum of 1 per acre	Liner material hand-tamped	Yes	½ lift thickness, if possible; none greater than lift thickness	4 to 6 inches
E	Frequency, per unit volume of fill, based on experience and liner material homogeneity	Modified triaxial, fixed-wall, and consolidation lab tests--per unit volume of fill, based on experience	--	--	--	3 to 6 inches or ½ lift thickness
F	Statistical approach	Lab tests--5 to 6 per acre	--	Yes	--	--
G	9 per acre of lift	Falling head field test at density test location--1 per acre of lift	--	Yes	--	--
H	1 per 2,000 ft^2 or at least 1 per lift	Lab test--1 per 10,000 to 20,000 ft^2 or every third lift	Liner material or bentonite hand-tamped	--	1 inch in upper lifts; may be larger in lower lifts	1 inch

(continued)

TABLE 5-8 (continued)

Source of information[a]	Minimum frequency of in-place moisture or density tests	Permeability testing	Sample or test hole filling method	Test fill recommended	Maximum clod size allowed	Maximum rock or root size allowed
I	1 per 2,500 ft²-minimum of 4 per lift or 1 per day--more as needed	Laboratory test--1 per 2,000 yd³ or 2 per lift--occasional field test	Bentonite	Yes	½ lift thickness	--
J	1 per 10,000 ft² or 1 per lift	Lab and field	--	Yes	½ lift thickness if possible; none greater than lift thickness; 1 to 3 inches for hand-compacted areas	--
K	Site specific, based on experience	Lab triaxial	Bentonite	Sometimes, especially for admixtures	3 to 6 inches, depending on compaction equipment; 1 inch in upper 1 foot of liner if overlain by FML; none greater than lift thickness	1 inch
L	1 per 1,000 yd³ or 1 per day	Lab constant head--1 per 25,000 yd³ or 1 per week	--	Yes	--	--
M	1 per 1,000 yd³ or 1 per day	Lab falling head--1 per 25,000 yd³	--	No	--	--
N[b]	1 per 1,000 yd³--1 per 200 yd³ if hand tamped	--	--	Yes	3 inches; 3/4 inch for blended materials	3 inches
O[c]	1 per 500 to 1,000 yd³	--	--	--	3 inches; 1 to 3 inches for hand-compacted areas	--
P[d]	1 per 2,000 yd³	Lab--1 per 16,000 yd³, field - 1 per 40,000 yd³	--	--	3 to 6 inches	--

[a] Sources A through M were design and geotechnical engineers with clay liner construction experience.

[b] U.S. Bureau of Reclamation, 1974.

[c] U.S. Department of the Navy, 1982.

[d] Ghassemi et al., 1983.

TABLE 5-9. RECOMMENDATIONS FOR CONSTRUCTION DOCUMENTATION OF CLAY-LINED LANDFILLS BY THE WISCONSIN DEPARTMENT OF NATURAL RESOURCES

Item	Testing	Frequency
1. Clay borrow source testing	Grain size	1,000 yd^3
	Moisture content	1,000 yd^3
	Atterberg limits (liquid limit and plasticity index)	5,000 yd^3
	Moisture-density curve	5,000 yd^3 and all changes in material
	Lab permeability (remolded samples)	10,000 yd^3
2. Clay liner testing during construction	Density (nuclear or sand cone)	5 tests/acre/lift (250 yd^3)
	Moisture content	5 tests/acre/lift (250 yd^3)
	Undisturbed permeability	1 test/acre/lift (1,500 yd^3)
	Dry density (undisturbed sample)	1 test/acre/lift (1,500 yd^3)
	Moisture content (undisturbed sample)	1 test/acre/lift (1,500 yd^3)
	Atterberg limits (liquid limit and plasticity index)	1 test/acre/lift (1,500 yd^3)
	Grain size (to the 2-micron particle size)	1 test/acre/lift (1,500 yd^3)
	Moisture-density curve (as per clay borrow requirements)	5,000 yd^3 and all changes in material
3. Granular drainage blanket testing	Grain size (to the No. 200 sieve)	1,500 yd^3
	Permeability	3,000 yd^3

Source: Gordon et al., 1984.

TABLE 5-10. ELEMENTS OF A CONSTRUCTION DOCUMENTATION REPORT

Major elements	Components
A. Engineering plans	• Completed sub-base elevations.
	• Final clay liner grades.
	• Top of drainage blanket grades.
	• Leachate collection lines, cleanouts, and manholes with spot elevation every 100 feet along the lines and at all manhole entrances and exits.
	• Drainage features.
	• All monitoring devices.
	• Spot elevations at all breaks in slope and on approximate 100-foot centers.
	• Document testing locations.
	• Other site information as appropriate.
B Engineering cross-sections	• Minimum of one east-west and one north-south through the completed area.
C. Comprehensive narrative	• Explaining how construction of the project was accomplished along with an analysis of the soil-testing data obtained in 1 through 3 above. This report should also include an appendix containing all the raw data from the field and laboratory testing.
D. Series of 35-mm color prints	• Documenting all major aspects of site construction.
E. Construction certification	• Should be certified by a registered professional engineer to have been completed according to the approved plans. Any deviations from the plans should be noted and explained.

Source: Gordon et al., 1984.

Statistical sampling methods for earthwork quality control (geostatistics) have been developed by the U.S. Department of the Interior and the U.S. Army Corps of Engineers (U.S. Department of the Interior, 1974; U.S. Department of the Army, 1977; Davis, 1966; Turnbull et al., 1966; Willenbrock, 1976). These documents include statistical methodology to establish degree of confidence for a testing program. Other discussions of statistical methods may be found in Winterkorn and Fang (1975), Selig (1982), Wahls et al. (1968), Jorgenson (1971), and Kotzias and Stamatapoulous (1975).

Figures 5-20 and 5-21 illustrate a simple, concise method of documentation of clay liner CQC statistics developed by Soil Testing Engineers of Baton Rouge, Louisiana. Figure 5-20 illustrates the sampling locations for QC tests of a clay-lined landfill cell. Figure 5-21 is a graphic presentation of the moisture/density/permeability data and clearly illustrates the statistical analyses performed on these data. These figures are presented as examples of good CQC documentation and statistical analysis.

5.3.4 Documentation

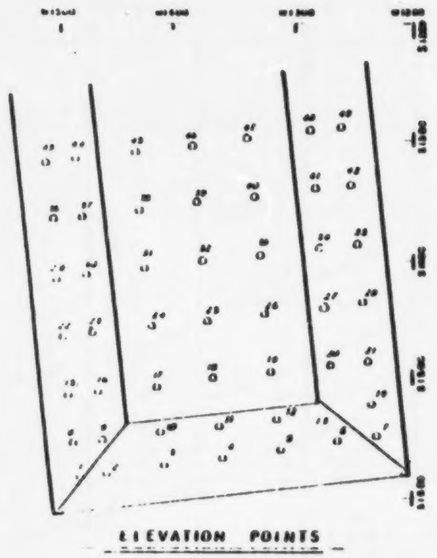
An effective CQA plan depends heavily on recognizing all construction activities that should be inspected and assigning responsibilities to CQA and CQC personnel for inspecting each activity. This is most effectively accomplished by documenting CQA and CQC activities and should be addressed as the fifth element of the CQA plan. CQC personnel will be reminded of the factors to be inspected and will note by signing required descriptive remarks, data sheets, and checklists that the inspection activity has been accomplished.

5.3.4.1 Daily Recordkeeping--

Standard daily reporting procedures should include preparation of a summary report with data sheets for supporting observation and testing and, when appropriate, for problem identification and corrective measures.

5.3.4.1.1 Daily Summary Report--A summary report, or project inspection diary, should be prepared daily by the CQA officer. This report provides the chronologic framework for identifying and recording all other reports. At a minimum, the summary reports should include the following information (Spigolon and Kelley, 1984):

- Unique identifying sheet number for cross-referencing and document control
- Date, project name, location, and other identification
- Data on weather conditions
- Reports on any meetings held and their results
- Unit processes, and locations, of construction underway during the time frame of the report

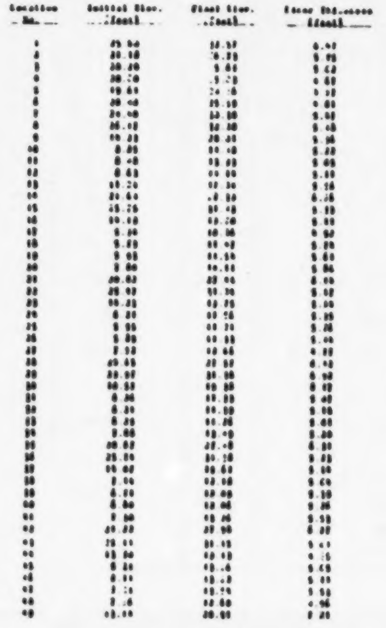


ELEVATION POINTS

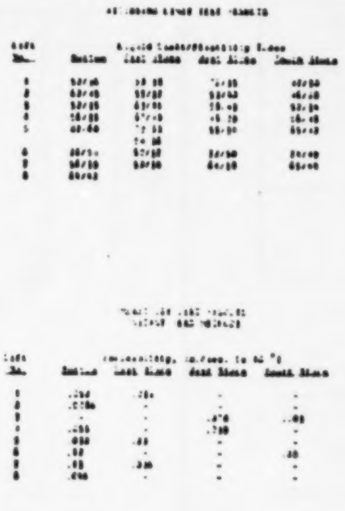
NOTE

Liner thicknesses given for points on slopes are perpendicular to slope face

Elevation data was corrected for slope angle in the case of such points



LINER THICKNESS DATA



LABORATORY TEST DATA

SUMMARY of RESULTS

THICKNESS DATA:

Number of data points = 49
Average thickness value = 5.5
Standard deviation = 0.711

COMPACTION DATA:

Number of tests = 148
Average compaction = 91%

ATTERBERG LIMIT DETERMINATIONS:

Number of tests = 30
Average values:
liquid limit = 59
plasticity index = 40
Standard deviation:
liquid limit = 13
plasticity index = 17

PERMEABILITY TESTS:

Number of tests = 14
Average value = 0.053×10^{-7} cm/sec

PERMITTING STUDY

SOIL INVESTIGATION BY
SOIL TESTING ENGINEERS, INC.
Baton Rouge, La. Lake Charles, La.

	BY	CKD		DRAWING NO.
	LGW	SJL		Q-7a

Note: Courtesy of Soil Testing Engineers, Inc., Baton Rouge, LA.

Figure 5-20. COC test location and data summary.

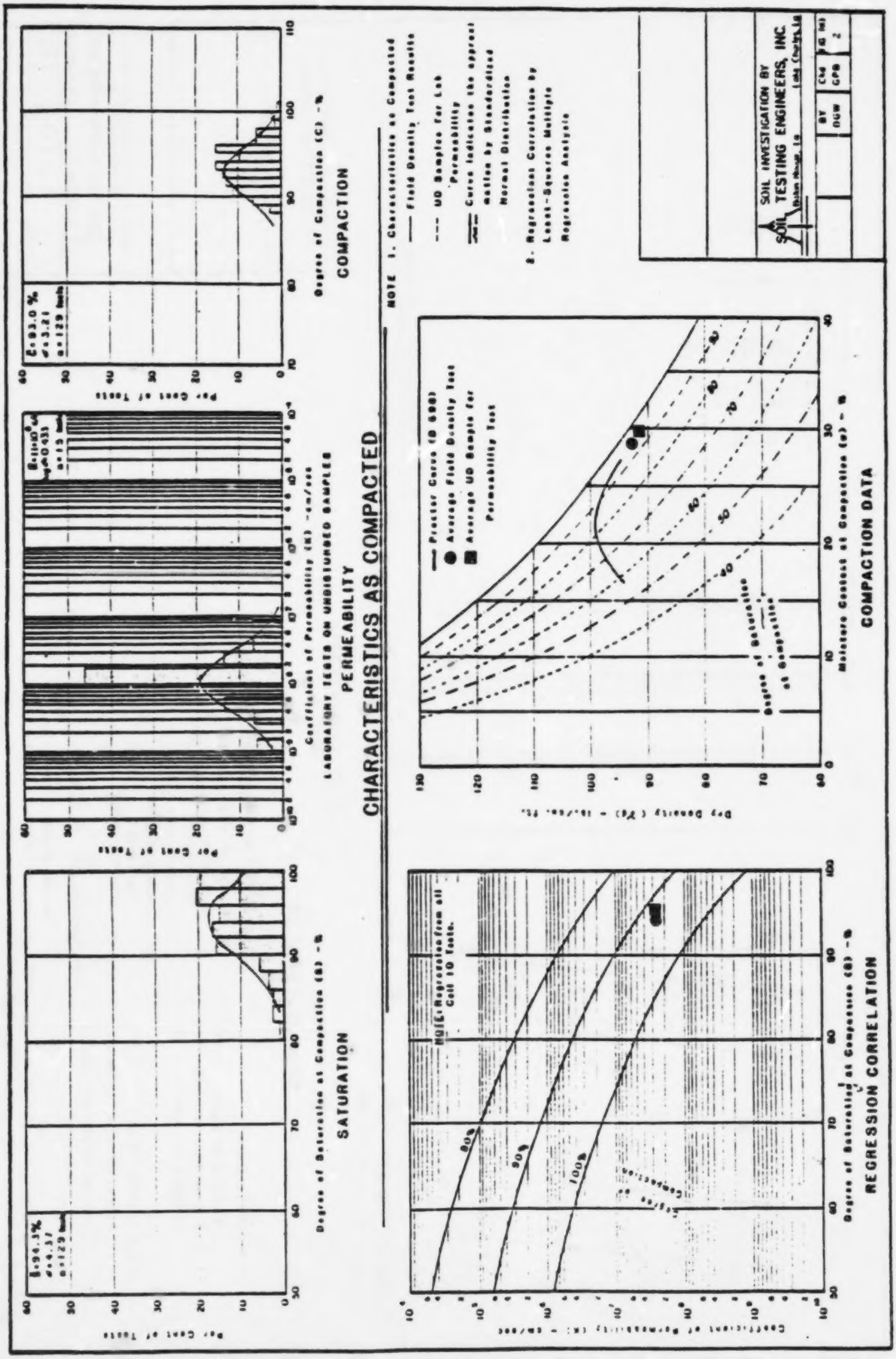


Figure 5-21. Statistical analysis of CQC test data.

Note: Courtesy of Soil Testing Engineers, Inc., Baton Rouge, LA.

- Equipment and personnel in each unit process, including subcontractors

- Descriptions of areas or units of work (blocks) tested and/or observed and documented

- Description of offsite materials received, including any quality verification (vendor certification) documentation

- Calibrations, or recalibrations, of test equipment, including actions taken as a result of recalibration

- Decisions made regarding approval of units of material or of work (blocks) and/or corrective actions to be taken in instances of substandard quality

- Unique identifying sheet numbers of observation and testing data sheets and/or problem reporting and corrective measures data sheets used to substantiate decisions described in the preceding item

- Signature of the CQC inspector and concurrence by the CQA officer.

Items above may be formulated into site-specific checklists and data sheets so that details are not overlooked.

5.3.4.1.2 Observation and Testing Data Sheets--All observations and field and/or laboratory tests should be recorded on appropriate data sheets. Required data to be included in a test report (data sheet) for most of the standardized test methods are included in the pertinent ASSHTO (1983) and ASTM (1985) standards. Examples of field and/or laboratory test data sheets are given in U.S. Department of the Army (1970, 1978) manuals and in Spigolon and Kelley (1984).

Due to their nonspecific nature, no standard format can be given for data sheets to record observations. Recorded observations may take the form of notes, charts, sketches, photographs, or any combination of these. Where possible, a checklist may be useful to ensure that no pertinent factors of a task-specific observation are overlooked.

Observation and testing data sheets should include at least the following information (Spigolon and Kelly, 1984):

- Unique identifying sheet number for cross-referencing and document control

- Description or title of the observation/test

- Location of the observation/test or location from which the sample increment was obtained

- Type of observation/test; procedure used (reference to standard method when appropriate)
- Recorded observations or test data, with all necessary calculations
- Results of the observation/test; comparison with specification requirements
- Personnel involved in the observation/test
- Signature of the CQC inspector and concurrence by the CQA officer.

Items above may be formulated into site-specific checklists and data sheets so that details are not overlooked.

5.3.4.1.3 Problem Identification and Corrective Measures Data Sheets--Problem reporting and corrective measures data sheets should be cross-referenced to specific observation or testing data sheets where the problem was identified. They should, at a minimum, include the following information:

- Unique identifying sheet number for cross-referencing and document control
- Detailed description of the problem
- Location of the problem
- Probable cause
- Method and time frame of locating the problem (reference to data sheets)
- Estimated duration of the problem
- Suggested corrective action
- Documentation of correction (reference to data sheet)
- Final results
- Suggested methods to prevent similar problems
- Signature of the CQC inspector and concurrence by the CQA officer.

In some cases, not all of the above information will be available or obtainable. However, when available, such efforts to document problems could help to avoid similar future problems.

The CQA officer should be made aware of any significant recurring nonconformances. The CQA officer will then determine the cause of any problems and recommend appropriate changes to prevent future recurrence. When this type of evaluation is made, the results should be documented.

Upon receiving the CQA officer's written concurrence, copies of the report(s) should be sent to the design engineer and the facility owner/operator for their comments and/or acceptance. These reports should not be submitted to the permitting agency unless they have been specifically requested. However, a summary of the data sheets along with final testing results and inspector certification of the facility may be required by the permitting agency upon completion of construction.

5.3.4.2 Photographic Reporting Data Sheets--

Photographic reporting data sheets may also prove useful. Such data sheets should be cross-referenced with observations or testing data sheets and/or problem identification and corrective measures data sheets. Photographic reporting data sheets should include the following minimum information:

- A unique identifying number for cross-referencing and document control
- The date, location, and weather conditions for the photograph
- Location and description of the work or work product
- Purpose of the photograph
- Signature of the photographer and CQC inspector.

These photographs will serve as a pictorial record of work progress, problems, and mitigation activities. They should be kept in plastic file sheets in the chronological order in which they were taken. The basic file should contain color prints; negatives should be stored in a separate file in chronological order.

A video recording of problem areas and/or conditions and of the completed installation of each soil component may also prove useful.

5.3.4.3 Block Evaluation Reports--

Each inspection block may have several quality characteristics, or parameters, that are specified to be observed or tested, each by a different observation or test, with the observations and/or tests recorded on different data sheets. At the completion of each block, these data sheets should be organized into a block evaluation report. These block evaluation reports may then be used to summarize all of the site construction activities.

Block evaluation reports should be prepared by the CQA officer and, at a minimum, should include the following information (Spigolon and Kelley, 1984):

- Unique identifying sheet number for cross-referencing and document control
- Description of block (use project coordinate system to identify areas and appropriate identifiers for other units of materials or work)
- Quality characteristic being evaluated; references to sections of specifications
- Sampling method; how it was established
- Sample increment locations (describe by project coordinates or by a location sketch on the reverse of the sheet)
- Tests and/or observations made (define procedure by name or other identifier; give unique identifying sheet number for observation test data sheets)
- Summary of test data (give block average and, if available, the standard deviation for each quality characteristic)
- Define acceptance criteria (compare block observation/test data with design specificatin requirements; indicate compliance or noncompliance; in the event of noncompliance, identify documentation that gives reasons for acceptance without specification compliance)
- Signature of the CQA officer.

5.3.4.4 Design Engineer's Acceptance of Completed Components--

All daily inspection summary reports, observation and testing data sheets, problem identification and corrective measures sheets, and block evaluation reports should be reviewed by the CQA officer and then submitted to the design engineer. The reports should be evaluated and analyzed for internal consistency and for consistency with similar work. Timely submission of these documents will permit errors, inconsistencies, and other problems to be detected and corrected as they occur, when corrective measures are easiest.

The design engineer should assemble and summarize the above information into a periodic design acceptance report. The reports should indicate that the materials and workmanship comply with design specifications and permit requirements. These reports should be included in project records and, if requested, submitted to the permitting agency.

5.3.4.5 Final Documentation--

At the completion of the project, the facility owner/operator should submit a final documentation report to the permitting agency. This report should include all of the design engineer's acceptance reports (i.e., periodic summaries of all daily inspection summary reports, observation and test data sheets, problem identification and corrective measures data sheets, and block evaluation reports), deviations from design and material specifications (with justfying documentation), and as-built drawings. This document should be prepared by the CQA officer and included as part of the CQA plan documentation.

5.3.4.5.1 Responsibility and Authority--The final documentation should reemphasize that areas of responsibility and lines of authority were clearly defined, understood, and accepted by all parties involved in the project. Signatures of the facility owner/operator, design engineer, CQA officer, CQC inspector, and construction contractor should be included as confirmation that each party understood and accepted the areas of responsibility and lines of authority and performed their function(s) according to the CQA plan.

5.3.4.5.2 Relationship to Permitting Agencies--Final documentation submitted to the permitting agency as part of the CQA plan documentation does not sanction the CQA plan as a guarantee of facility construction and performance. Rather, the primary purpose of the final documentation is to improve confidence in the constructed facility through written evidence that the CQA plan was implemented as proposed and that the construction proceeded according to design plans and specifications.

5.3.4.6 Storage of Records--

During the construction of a hazardous waste land disposal facility, the CQA officer should be responsible for the facility records, including the originals of all the data sheets, summary reports, and block evaluation reports; the design engineer's acceptance of completed components reports; and facility drawings. With the CQA officer in charge of the facility construction records, any documentation problems should be noted and therefore remediated quickly. Once the facility construction is complete, the document originals should be stored by the owner/operator in a manner that will allow for easy access. An additional copy should also be kept at the facility if this is in a different location from the owner/operator's files. A final copy should be kept by the permitting agency in a publicly acknowledged repository.

5.4 CLAY LINER DESIGN AND CONSTRUCTION: PROBLEMS AND PREVENTIVE MEASURES

Table 5-11 lists common problems encountered during clay liner design and construction, their probable causes, and suggested solutions. This table was compiled from information assembled from a wide variety of sources, including literature surveys, case studies, and interviews with experts in the field. This table summarizes information presented in this chapter.

TABLE 5-11. POTENTIAL CLAY LINER DESIGN AND INSTALLATION PROBLEMS AND PREVENTIVE MEASURES

Problems	Cause	Preventive measures
Sidewall slump and collapse	· Improper characterization of soil strength profile that results in improper sidewall design.	· Properly characterize subsurface conditions.
	· High inward hydraulic pressure on sidewalls (sites below water table).	· Design a more gentle sidewall slope (depending on shear strength of foundation soil). · Reduce hydraulic head by: -- Installing slurry wall around site perimeter to cut off groundwater. -- Trenching and pumping around site to cut off groundwater. -- Pumping from wells to lower local groundwater table.
Bottom heave or rupture	· High inward hydraulic pressure on bottom (sites below water table).	· Control depth of excavation to lower head potential. · Reduce hydraulic head by slurry wall, pumping, or other technique. · Fill landfill before heaving occurs.
Accumulation of water in landfill during construction	· Rainfall	· Cover site with inflatable dome (reduces leachate treatment requirements for continuous operation facilities). · Seal roll liner at end of construction day to ensure proper runoff of precipitation into sump.
	· Seepage into site (sites below water table)	· Reduce hydraulic head in surrounding soil. · For all infiltration: -- Operate leachate collection system to remove water. -- Design leachate collection system or detection system (between liners) to handle extra water input.
Drying and cracking of clay liner, greatly increasing permeability	· Desiccation	· Do not construct during extremely hot, dry periods. · Wet down liner during dry periods. · Cover liner with plastic sheet or soil layer if liner construction is interrupted. · Do not leave liner exposed prior to waste emplacement or leachate collection system installation.

(continued)

TABLE 5-11 (continued)

Problems	Cause	Preventive measures
Loss in liner density (increased permeability)	· Freeze/thaw	· Do not construct during winter in cold climates · Cover liner with soil blanket or other insulation material when cold weather is anticipated.
Reduction in clay workability	· Low temperatures	· Increase compactive effort. · Cease construction till spring.
Erosion of upper liner after construction (liquid impoundments)	· Wave erosion	· Place rip-rap on lagoon sidewalls extending from below liquid level into the freeboard area.
Visible partings between liner lifts and increased permeability parallel to lifts	· Liner lifts not properly tied together	· Scarify or disk lower lift prior to installing next lift. · Ensure moisture content of last lift and lift being installed are the same.
Pockets of high-permeability material (e.g., sand and gravel) in liner material	· Heterogeneous clay liner material	· Closely inspect liner material at borrow site or as it is being installed and reject coarse-grained material.
Leakage around designed liner penetrations	· Improper sealing around penetrating objects	· Avoid liner penetrations in design. · Seal properly around penetrating objects.
Leachate collection system clogging	· Sediment entering into systems	· Cover system with geotextile or graded soil layer.
Leachate collection system damage during waste emplacement	· Inadequate management of personnel	· Initiate personnel training program. · Cover system with geotextile or soil cover prior to waste management. · Design system to minimize protrusions (manholes, etc.) in waste emplacement areas.
Areas of high permeability	· Substandard compaction	· Conduct proper CQA and CQC, including: - Monitoring for material variability - Observation of compaction operations - Moisture/density/permeability tests.

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CHAPTER 6

FAILURE MECHANISMS

This chapter discusses failure mechanisms that can affect performance of clay liners at waste disposal facilities. The failure mechanisms described include:

- Desiccation cracking
- Slope instability
- Settlement
- Piping
- Penetration
- Erosion
- Freeze/thaw cycling
- Earthquakes
- Scouring
- Design or construction errors.

When available, laboratory and field studies of these failure mechanisms are presented along with actual site data. Special design considerations that may help to control these types of failure mechanisms are discussed in Section 5.1.3.4 of this document.

6.1 DESICCATION CRACKING

6.1.1 Description

Cracking, a characteristic of clay-rich soils, results from volume changes in the clay structure or from stresses imparted to the soil mass. Desiccation cracks result from wet/dry cycling, which can disrupt the cohesive structure of the clay. Syneresis cracks result from a chemical reaction that causes shrinkage and dewatering of the colloidal clay material due to aggregation of particles by physicochemical attraction. Clay minerals that expand in water can undergo expansion or shrinkage caused by (1) replacement or removal of water by other solvents, (2) cation exchange, or (3) increasing ionic strength at the clay/liquid interface (Griffin et al., 1984). The amount of volume change depends upon the clay mineral type, the arrangement of the clay particles, the size and surface area of the clay particles, and the kind and proportion of cations adsorbed on the clay (Brown and Anderson, 1980). Stress cracks result from stress applied to clay soils through seismic activity, subsidence, or human action (e.g., compaction).

The phenomenon of crack formation is closely related to clay mineralogy. Certain clay minerals are much more prone to shrinking and crack formation. Montmorillonite (bentonite) clay is the most notorious for shrinking and cracking, and soil engineers in the Great Plains and Gulf States continually face problems with these clay soils in road construction and building foundations (Lutton et al., 1979). For example, montmorillonite can have a 200-percent difference in volume between the hydrated and dehydrated state. When water is removed the clay undergoes three-dimensional shrinkage and, when the water removal rate is not uniform, cracks form in the wetter portion. Disruption of clay barriers also may occur during swelling or uptake of water. A barrier that has swelled and heaved may lose its integrity upon heaving or it may shrink or crack later when the water is removed (Brown and Anderson, 1980).

Volume changes also may occur by chemical desiccation when interlayer water is extracted or displaced by an organic fluid. Extraction or displacement of water changes the interlayer spacing, which may result in a volume change. If the fluid lacks a large dipole moment or cannot form a hydrogen bond, interlayer spacing may decrease since fewer layers of organic molecules are retained. If the intruding fluid has a higher affinity than water for the clay's surface, interlayer spacing may decrease greatly since some of the more tightly bound water is removed.

6.1.2 Studies of Cracking

Chemical desiccation (i.e., the extraction or displacement of interlayer water by chemical species) is discussed in detail in Chapter 4. This section discusses studies of cracking from wet/dry cycling (thermal desiccation) in sites where the cracking phenomenon has been observed.

Hawkins and Horton (1967) investigated the necessary soil cover and optimum thickness to protect a bentonite layer that had been placed as a protective cover for buried radioactive waste. The bentonite layers protected (covered) by 2 feet of soil did not crack. Layers protected by less topsoil formed cracks that were resealed by swelling, but some of the overlying soil became entrapped in the cracks. After various shrink/swell cycles, it is possible that permeable channels may develop through a bentonite layer. Kays (1977) also noted that desiccation cracks in reservoir linings made of bentonite may not reseal entirely upon rewetting.

Rollin and Dylla (1970) investigated the effectiveness of bentonite seals for water retention in the arid West. One seal was constructed as a 3-inch layer (12.5 percent bentonite in sand) covered with 6 inches of earth. A second seal was a pure layer of bentonite 0.5 inch thick covered with 6 inches of earth. The seals were exposed to a series of wet/dry cycles over 6 years. The seals retained their initial effectiveness during the first 3 years, but their effectiveness decreased by roughly 10 percent per year for the last 3 years of the study. Rollin and Dylla estimate a seal life under extreme conditions of 8 to 12 years; however, they do not indicate the percentage loss of effectiveness they assume will terminate the life of the seal. Note that the burial depth was 6 inches, which is inadequate based on the previously cited study.

Drying tests were performed on a polymer-treated bentonite, Volclay SLS®, by its manufacturer (Hughes, 1975). The bentonite was packed into a fixed-wall permeameter and covered with several inches of soil. The permeability to leachate (composition unknown) was found to be 1.5×10^{-7} cm/s. The material was then subjected to drying cycles with surface temperatures of 150 °F (in the cover soil) for 1 month and again exposed to leachate. After a drying cycle, cracks extended 0.25 to 1 inch into the cover soil. The permeability after three drying cycles was 1.0×10^{-7} cm/s, which was essentially the same as the start of the experiment.

Permeabilities were measured after the leachate had contacted the dried-out soil cover for 4 hours to allow the soil time to absorb water to compensate for the natural moisture loss. The manufacturer claimed that it was not visually obvious that the leachate had penetrated the seal. However, it may have been more meaningful to start measuring the permeability when the leachate was introduced because the critical period in the field occurs when the cracks are still open (Pertusa, 1980).

Desiccation cracks were observed in the cover system of the West Valley, New York, disposal site for low-level radioactive waste (Prudic and Randall, 1979). These cracks propagated downward, and subsidence cracks were found that propagated upward. The cracks increased the cover permeability and resulted in the overflow of leachate, or the "bathtub" effect. Climate records were examined, revealing that crack formation correlated with the two driest periods at the site.

Daniel (1983) analyzed data on several leaking clay liners in Texas. He concluded that desiccation of the clay liner between completion of construction and commencement of operations allows the clay to become severely cracked and results in higher permeabilities. Daniel (1983) also concluded that higher permeabilities may result from nonuniform moisture distribution in the clay, resulting in clods with wet surfaces and dry, cracked interiors. This problem is caused in part by (1) inadequate breakup of large clods during compaction, (2) uneven distribution of water by large water trucks, and (3) inadequate time for water to penetrate the soil.

A case study of a failed liner system caused by thermal desiccation (case study I) is presented in Chapter 7 of this document. A summary of this case study is presented below.

Problems that occurred at site I resulted from desiccation cracks in the facility's clay liner. Liner construction was completed in the fall of 1980; however, it was left unprotected and unfilled until late the following spring. During this time, desiccation cracks formed in the liner, increasing its permeability. When the pond was finally filled, waste migration into the leak detection system occurred.

6.2 SLOPE INSTABILITY

Some landfills and impoundments are constructed above or partially above ground with retaining walls to prevent the lateral spread of potentially harmful wastes. Failure of the sidewall liner may release contaminants directly into the environment or allow contaminant movement into surrounding zones of permeable soil.

6.2.1 Description

The soil in a sloping liner has an internal shear stress. If site configuration is such that the overburden forces due to gravity become too large compared to the shear strength of the liner, the slope is no longer stable and a slide occurs. The site configuration and the shear strength of the soil are the key for determining the safety factor for design of stable slopes.

6.2.2 Discussion of Slope Instability

The mechanisms involved in slope instability and slides vary greatly with soil type and strength. Only a few of the more important mechanisms for slides are discussed here. For example, shallow slides involve only the upper few feet of the slope and can occur when flakes are pushed out by water or bonding is weak between two soil types. Local slides occur when the shear strength is insufficient for a limited part of the site. Local slides usually do not spread and can be avoided with use of a slightly flatter slope. Large slides include several different phenomena where a whole valley or parts of it are involved and are usually considered natural disasters. Failure of a clay liner sidewall can occur as slippage of the whole compacted layer over the undisturbed soil or bedrock. Another mechanism is the creep of very cohesive saturated clays; by this mechanism, tension can be generated in a sidewall slope and cracks perpendicular to the slope can enhance failure. Waste consolidation can uncover sidewall slopes, relieve lateral pressures, and create an unstable condition (U.S. EPA, 1980). For sites with an earth barrier over a synthetic membrane, a slide may develop if the clay slips over the membrane's smooth surface.

The probability of a local slide is greatest during construction if the excavation is deep and the sidewall slopes are not designed properly. Compaction of the liner on the sides can be difficult and may be insufficient. If the soil in the sidewall slope is saturated, the compacted layer may slide. Once the waste is put in place, the slope stability may improve and the possibility of shallow slides decrease.

If the landfill is large and placed on a slope, loading of the landfill can increase the weight considerably, which might be a major factor in triggering a large slide. However, waste placed at the foot of the slope increases the stability of that slope once the fill is completed.

Changes in soil moisture conditions also must be considered. For example, frozen soils thaw from the top, and thaw water trapped over frozen soil may form a shallow, saturated zone that is susceptible to sliding. Similarly, experience indicates that shallow sliding sometimes occurs in clay embankments after heavy rainfall. This instability presumably results from rapid saturation of a shallow zone previously made porous and weak by formation of drying cracks (Lutton et al., 1979). Compaction is necessary because it reduces porosity and increases shear resistance by increasing the interlocking of soil grains.

When landfills or impoundments are sited below the water table, sidewall slope failure can result from inward hydraulic pressure on the liner. This is a problem only during construction and early operation; once the landfill or impoundment is filled with waste, outward pressure from the waste balances the inward hydraulic pressure.

6.3 SETTLEMENT

6.3.1 Description

Differential settlement of the bottom liner may lead to cracks, increasing the volume of leachate escaping the site. However, the problem should be minor if the liner is compacted, sufficiently thick, and placed on a uniform foundation. The subsoil will show some settlement, but no adverse impacts are expected if the settlement is uniformly distributed and moderate. If the subsoil settlement is large, tension cracks in the side slope of the liner can result.

Extreme subsidence leading to a catastrophic collapse of landfill or lagoon liner can occur through sinkhole formation in areas underlain by Karst geology. The only way to control this type of failure is to avoid siting facilities in regions of Karst terrain.

6.3.2 Studies of Settlement

A study was also made at the West Valley, New York, site that was used in the 1960's for disposal of low-level radioactive waste similar to that described for the Sheffield site (Prudic and Randall, 1980). The cover material consisted of approximately 10 feet of glacial till that was composed of 35 percent silt, 35 percent clay, and about 30 percent sand and stone. This material was covered with 1 foot of topsoil and then vegetated.

Problems developed in the 1970's with the formation of cracks and a rise in the water level in the trenches ("bathtub" effect). The study identified two major causes of the cracks: (1) subsidence of the waste causing cracks that propagated upward and (2) desiccation cracks that propagated downward (Prudic and Randall, 1980). The usual disposal practice for low-level radioactive waste sites is not to compact or cover the wastes each day to avoid contaminating equipment and personnel. This practice was followed at the West Valley site (Kelleher and Michael, 1973).

Buelt and Barnes (1981) performed a laboratory study of the effect of subsidence on several liner materials that were candidates for lining waste piles containing uranium mill tailings. The soil liners consisted of soil with 10 percent clay in the form of sodium bentonite, Saline Seal 100®, and GSR-60® (American Colloid Company). The liners were compacted into a cylinder, sand was placed on top to simulate the tailings, and a water-filled bladder was placed under the liner to simulate subsidence. Accelerated aging was carried out by passing several pore volumes of high-calcium leachate through the liner to accelerate the ion exchange rate of calcium and sodium. After 50 days, a subsidence of 3.5 percent was induced by emptying the bladder.

The soils with Saline Seal 100® and GSR-60® had high initial permeabilities (10^{-6} to 10^{-5} cm/s) because (1) they were not wetted prior to compaction, (2) their granular nature inhibited mixing into the smaller pores of the soil, and (3) the subsidence bladder made it difficult to obtain the desired compaction.

The permeability of the Saline Seal 100® liner decreased from its originally high value and reached a plateau of 6×10^{-6} cm/s until the subsidence test. When subsidence was induced, permeability increased as the liner was disrupted. The permeability then slowly decreased, indicating that this liner exhibits some self-healing characteristics.

The permeability of the GSR-60® liner remained constant at 3×10^{-6} cm/s until subsidence was induced and then increased significantly. Again, the clay demonstrated a tendency to seal the liner following a disruptive incident by permeability reductions following the subsidence test.

The permeability of the sodium bentonite liner increased initially and then decreased to about 10^{-7} cm/s. The increase may indicate Na-Ca ion exchange early in the test. The permeability of this liner did not increase when subsidence was induced. The authors observed that the sodium bentonite was better distributed in the soil because of its small particle size and thus produced a more stable liner. The sodium bentonite liner was judged to be the most effective of the clay liners in this laboratory test. These results may or may not be indicative of liner performance in a field environment.

6.4 PIPING

6.4.1 Description

Piping is a form of internal soil erosion that occurs below the ground surface. Fine-grained soils such as clay contain small particles (1 to 2 microns in diameter) that are bound together within the soil matrix. As water seeps through the soil matrix, some of the fine clay particles can be detached from the soil matrix and can migrate into a coarser, more porous layer below the clay. As smaller particles erode, resistance to flow decreases and progressively larger particles can be removed from the soil matrix. As these particles are removed, the internal erosion increases and tends to form an underground flow channel or "pipe" that moves upstream; i.e., the "pipes" progress and grow larger in the opposite direction of the seepage flow (Bowles, 1979). For a clay barrier with downward infiltration, this means that the "pipes" initiate on the underside of the liner and progress upward. Piping can be caused by excessive seepage into the clay or by chemical dissolution of the clay by a reactive fluid.

6.4.2 Studies of Piping

A literature review on piping was performed by Brown and Anderson (1980) and is summarized here to provide insight into the piping mechanism and its effects.

Soil grains are dislodged and eroded away when the hydraulic gradient at the exit of a percolating soil mass is large enough to overcome the forces of interparticle bonding. Piping is more likely for soils with low structural stability or dispersive soils where the soil-colloid bond strengths are low compared to the energy of the water flowing through the soil. Some of the other factors that affect piping include exchangeable sodium percentage, soil cracks, low permeability, hydraulic gradients, chemical composition of the eroding water, and water content at compaction (Brown and Anderson, 1980).

The early stages of piping are associated with vertical contrasts in the structure and permeability of soils and with shifts in soil pore size distribution toward macropores (Brown and Anderson, 1980). Piping is also accelerated by the dissolution of the clay pore walls, and eventually the matrix between pores, by a reactive fluid. Clays, which contain large quantities of silica and alumina, are susceptible to dissolution by both acids and bases. Acids solubilize alumina, iron, alkali metals, and alkaline earths, while bases solubilize silica (Brown and Anderson, 1980).

Effects of piping have been observed on dams, where seepage of reservoir waters has caused dispersive piping and eventual channeling all the way through these earth structures. Although piping failures have occurred in earthen dams, the dams often are exposed to very high hydraulic gradients that are not expected in a properly constructed and operated disposal facility. Failure usually occurs through collapse of the pipe roof, which rapidly erodes as a result of the high water velocity. This mode of failure may conceal the fact that piping was the real cause of the disaster (Bowles, 1979). Channeling occurred in soils with a local permeability of 1×10^{-5} cm/s. Differential solution and subsequent leaching, especially with sediments containing calcium carbonate, resulted in formation of channels, pot holes, and cavities (Mitchell, 1976). In this respect, dissolution seems to be in some circumstances a precondition for piping.

Cedergren (1977) reported that differential leaching of limestone, gypsum, and other water-soluble mineral components can lead to development of solution channels that get larger with time and substantially increase permeability. He warned against underestimating the importance of minor soil and geologic details on the permeability of soil formations because they cause the majority of failures in dams, reservoirs, and other hydraulic structures. Cedergren (1977) concluded that most failures caused by excessive water seepage can be placed in two categories:

- Those that are caused by soil particles migrating to an escape exit, causing piping and erosional failures

- Those caused by uncontrolled seepage patterns that lead to saturation, internal flooding, and excessive seepage.

In a study of the variables affecting piping, Landau and Altschaeffl (1977) noted a strong interaction between the chemical composition of the eroding water and compaction water content. Ion concentration seemed to have little effect on soil piping susceptibility for mixed illitic and kaolinitic clay loam compacted dry of optimum. Optimum refers to the water content at maximum density as determined in the Proctor Density Test (see Chapter 3). For the same soil compacted wet of optimum, soil piping susceptibility was highly related to ion concentration in the eroding water. When low-ion-concentration eroding water is combined with wet-of-optimum compaction, Landau and Altshaeffl (1977) reported low resistance to internal erosion. These results indicate that an evaluation of leachate/liner interactions for liners compacted wet of optimum may be useful to predict resistance to piping.

Some piping involves the slaking of soil particles. Slaking is defined as the disintegration of unconfined soil samples when submerged in a fluid. Moriwaki and Mitchell (1977) investigated the dispersive slaking of sodium- and calcium-saturated kaolinite, illite, and montmorillonite. All the clays slaked by dispersion when saturated with sodium, with the process proceeding faster with sodium-kaolinite and sodium-illite. Sodium-illite swelled slightly, while dispersion of sodium-montmorillonite was preceded by extensive swelling. Sodium-kaolinite underwent no visible swelling while dispersing. For the calcium-saturated clays, illite dispersed much more slowly, while the dispersion rate increased for kaolinite and montmorillonite. Calcium-kaolinite was thought to disperse faster because of its higher permeability relative to sodium-kaolinite. Sodium-montmorillonite was thought to disperse slowly because the large degree of swelling it underwent would lower permeability, thus slowing water entry and retarding dispersion.

Compaction decreases the electrolyte content of expelled interlayer water (Rosenbaum, 1976). Such a lowering of fluid electrolyte concentrations in sodium-saturated clays may cause substantial swelling and dispersion (Hardcastle and Mitchell, 1974). This dispersion causes particle migrations. If the fluid-conducting pores are large enough to transport these dispersed clay particles, permeability increases and soil piping may result (Aitchison and Wood, 1965).

Piping initiates on the underside of a clay liner, where clay particles can migrate into a substrata with larger pore diameters. The soil pipe then progresses upward through the clay liner until it finds an opening into the waste impoundment. Clay particles have been shown to migrate through porous media containing less than 15 percent clay (Hardcastle and Mitchell, 1974). Consequently, clay liners underlain by soils containing less than 15 percent clay may be susceptible to soil piping.

6.5 PENETRATION

6.5.1 Description

Penetration of an earth barrier may result from operating equipment on the barrier, burrowing animals, root penetration, or vandalism. Once waste has been placed on the clay liner, the potential for damage due to one of these mechanisms is reduced. Designed penetrations such as manholes and leachate collection or detection system pipes, which are common at clay-lined hazardous waste disposal facilities, provide another mechanism for failure. Lack of adequate compaction or sealing around such penetrations may provide a pathway for leachate migration. Improper operation of mechanical equipment during waste placement operations may displace penetrating pipes, which in turn may damage the clay liner.

In arid regions, the danger of root penetration appears to pose more of a problem than does penetration by burrowing animals. Mammals in an arid region generally do not burrow more than 2 to 3 feet, with 5 feet being an overall maximum depth. Spade foot toads may penetrate to a depth of 6 feet. Root penetration for mesquite and tamarisk, which typically are found in arid regions, may range from 30 to 50 feet in alluvial soil to 2 to 3 feet on a rocky slope (Pertusa, 1980).

In rolled-earth dams, animals have created passageways for water to penetrate, which has damaged structural integrity. Similar damage can be expected in waste disposal facilities, where such holes may increase water percolation (Lutton et al., 1979).

6.5.2 Studies of Penetration

Rodents are a problem at some sites, particularly for municipal solid waste, and are best controlled by professionals (Noble, 1976). An extermination program is most effective when coupled with measures to eliminate sources of shelter and food.

A double-lined aeration lagoon recently was subjected to a hurricane. The winds from the storm caused the areator, which penetrated the clay liner, to sway. This movement resulted in the failure of the clay liner surrounding the penetration. This failure was confirmed when liquid from the pond was discovered in the leak detection system.

6.6 EROSION

6.6.1 Description

When raindrops strike bare soil at a high velocity, they shatter soil granules and clods and detach particles. Sheet erosion follows where detached soil particles are transported uniformly downslope. Rill erosion develops in channels only inches in depth when soil particles are detached by concentrated flow and by slumping of undercut sidewalls. The capability of runoff to detach soil increases approximately as the square of the shear stress. Consequently, rill erosion increases with increasing slope and channel length, while sheet erosion continues more uniformly between the rills; relative contributions to total soil loss differ with soil and surface conditions (Lutton et al., 1979).

6.7 COLD CLIMATE OPERATIONS (FREEZE/THAW CYCLING)

In Alaska, the northern tier of States, and in the mountainous regions, daily mean air temperatures fall below freezing frequently for days or even weeks during the winter. The extreme depth of frost penetration ranges from 0 to 6 inches in the southern United States to 60 to 70 inches in the northern United States. Average depths of frost penetration range from 0 to 3 inches to 20 to 60 inches for the same regions (Jones et al., 1982).

When air temperatures are below freezing, frost penetrates the ground surface, causing the upward migration of soil moisture. When this moisture contacts the frostline, ice may form. Various theories explain this phenomenon by means of capillary flow, suction force, thermodynamics, or vapor transfer (Young and Warkentin, 1966; Jumikis, 1962). The freezing effects upon soil mainly depend on the soil's chemical and physical properties and moisture conditions as well as on the freezing rate.

Two types of freezing phenomena occur in soils. Closed-system freezing in a soil occurs when no source of water is available, beyond that originally present in the soil voids, during the freezing process. This type of freezing usually results in very thin or nonexistent ice lenses. Open-system freezing, on the other hand, occurs when water in addition to the original soil pore water is available close to the freezing zone. In this case, ice lenses are formed in certain types of soils and heaving of the ground surface occurs. Closed-system freezing has been considered less important than open-system freezing because it generally does not result in frost heave. However, closed-system freezing has important effects on some soil liners for canals and reservoirs and on earth embankment dams (Jones, 1981).

Chamberlain and Gow (1978) studied the effects of freeze/thaw on the permeability of four fine-grained soils. Freezing and thawing cause significant structural changes in consolidated clay slurries, which in turn cause large increases in vertical permeability. The increase was greatest for the soil with the largest plasticity index and, in general, the increase was smaller at the highest applied stress levels. For soils where clay particles predominate, the increased permeability occurs as a result of the formation of vertical shrinkage cracks.

Wang and Roderick (1971) studied the freezing behavior of three different samples of compacted, fine-grained soils during closed- and open-system freezing. The soil samples ranged from 100 percent silt to a mixture of 33 percent silt, 57 percent clay, and 10 percent sand. The plasticity indexes ranged from 4 for the former to 21 for the latter soil sample. Other test variables included dry density and degree of saturation. Test results indicated that frost heave for the closed-system samples was much less than for the open-system samples partially because ice lenses in the open-system samples were much larger than in the closed-system samples.

The effect of closed-system freezing on canal and reservoir liners and earth embankments was studied in both the field and laboratory by Jones (1981). This study produced the following conclusions:

- In compacted fine-grained soil susceptible to frost action, closed-system freezing progresses inward from the soil surface at a slow to moderate rate. Either in a laboratory test specimen or in a field structure, the following takes place: (1) moisture in the soil voids migrates toward the cold surface, (2) soil density near the surface decreases, (3) density in a zone at some depth in the soil increases, (4) thin ice lenses may or may not form, and (5) under certain soil and freezing conditions overall shrinkage of the soil may occur.

- For some compacted soils in canal linings, an increase in density at depth, as well as a decrease near the surface of the lining, can be attributed to closed-system freezing.

- The closed-system type of freezing that occurs below the surface of an earth embankment when construction is halted by cold weather may significantly affect soil moisture and density. This will require the removal and/or recompaction of the upper zone of soil that has a density below the specification limits.

- The depth to which frost penetrates in soil with resulting frost action is determined mainly by cumulative, below-freezing temperatures; degree of shading; snow or other surface cover; and thermal properties of the soil. Frost penetrates deeper in soil with significant amounts of gravel because the thermal conductivity of gravel is higher than that of finer soil.

- Because of the higher thermal conductivity of solid bedrock compared to soil, it is possible that closed-system freezing could occur in compacted embankment soil in contact with a bedrock abutment at depths below those in the embankment and to some distance from the abutment. This could cause a zone of soil with higher moisture content and lower density near the abutment contact that, if not corrected, might produce a condition conducive to piping of the soil.

- A loose soil cover effectively reduces frost action in a compacted embankment.

- In certain soil and climatic conditions, closed-system freezing can cause soil shrinkage and the formation of cracks that might be detrimental to soil barriers for retaining water.

Other winter problems at landfills may include frozen waste, snow interference, frozen cover soil, and slope sliding. Frozen wastes are difficult to distribute and compact, and subsequent thawing of frozen waste may have an indirect, disruptive effect on the cover soil. Snow interference may result in snow and ice being incorporated into the cover soil and waste. Allowance should be made for this additional moisture in the disposal facility. Unfrozen cover soil may be difficult to supply during severely cold weather, so the cover may not be compacted to the extent desired by the design. Frozen ground conditions may be detrimental to the stability of landfill slopes during thawing. A layer of saturated soil with little or no strength may develop to a depth of a few feet over a frozen base. Such a development can create an unstable zone that may slide easily.

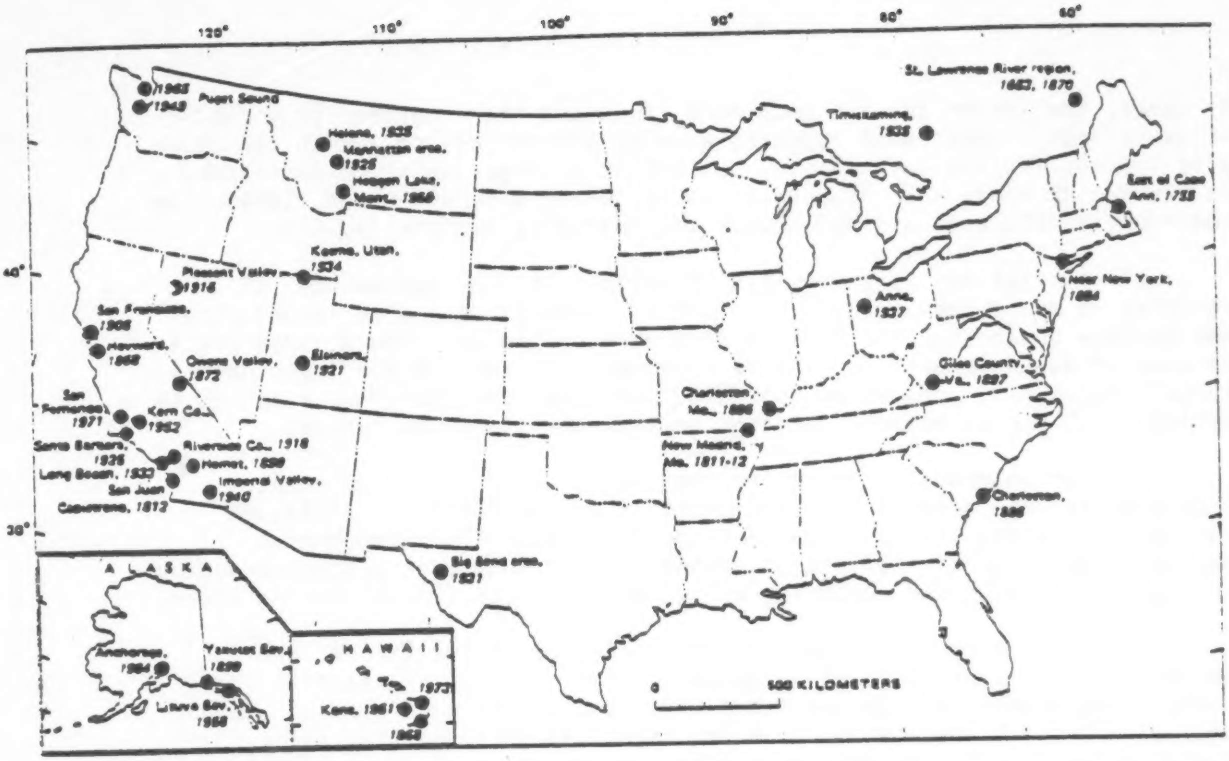
6.8 EARTHQUAKES

Except in California, current technology for the design and construction of waste disposal facilities has not addressed methods for increasing the resistance to failure from earthquakes. In the eastern United States, earthquakes, although much less frequent than in the western States (particularly California), are potentially damaging to a much larger area. This is because the subsurface conditions in the East result in propagation of ground motion over a much wider area. The location of past destruction earthquakes in the United States is illustrated in Figure 6-1. In Figure 6-2 areas of comparable damage are compared for the 1811 New Madrid, Missouri, earthquake and the 1906 San Francisco earthquake. Although the New Madrid earthquake was lower in magnitude, it resulted in damage to a much larger area of the country than the San Francisco event, because of differences in subsurface geology in the two areas.

The probability of failure from earthquakes is relatively low in most areas of the country and has received a low priority in design considerations. However, a methodology exists for designing earthworks to withstand earthquake-induced ground motion. This methodology originally was developed for dams and public works projects in California. The Nuclear Regulatory Commission (NRC) has considered seismic-resistant design for their low-level radioactive waste facilities. Chapter 5 of this document further discusses designing a facility for ground acceleration.

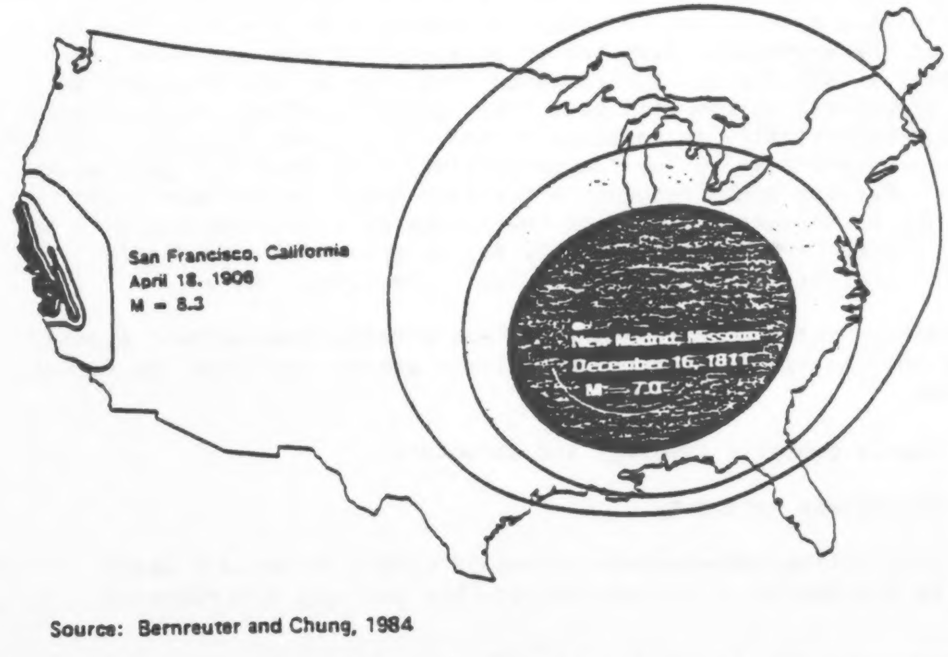
Three major earthquake-induced failure mechanisms exist for clay-lined hazardous waste facilities. These are:

- Rupture along faults
- Soil instability (e.g., liquefaction, soil failure, and large settlements)
- Deformation.



Source: Hays, 1980

Figure 6-1. Location of past destructive earthquakes in the United States.



Source: Bernreuter and Chung, 1984

Figure 6-2. Differences in propagation of damage for eastern and western earthquakes.

Of these, the latter two are much more important than rupture; only facilities on faults experience rupture, whereas ground shaking, which can cause soil instability and deformation, can affect a large number of facilities in the region where the earthquake occurs (Chung and Bernreuter, 1984). In addition, earthquakes are not always manifested by surface faulting.

Soil instability is a potentially serious failure mechanism. If a facility is diked above ground, failure of these dikes due to liquefaction can produce a catastrophic failure of the retaining wall and a large-scale release of waste into the environment (especially for surface impoundments). Large settlements, induced by ground motion under the facility, also can be serious. If the settlement is great enough, the liner can rupture.

Deformation results from a facility response to ground movement. Ground motion can distort the configuration of the facility. Although deformation is not likely to cause slope failure or liner breachment, damage to the leachate collection system or leak detection system can occur through the bending and rupturing of collection pipes and collection risers.

A facility also may be damaged indirectly by earthquake failure in nearby structures or geologic formations. For instance, a landfill or surface impoundment may successfully withstand ground acceleration without damage, only to be damaged by a nearby structure or slope that fails. Evaluation of seismic hazards at a site must therefore include all structures and geological features that could fail and damage the facility in question.

The degree of damage to a facility from an earthquake depends upon how well the facility was designed to resist ground acceleration and how much ground acceleration occurs at the site. Potential damage from ground acceleration at a site is not necessarily reduced with distance from the epicenter of the earthquake even though acceleration and amplitude are higher closer to the fault. The response frequency of the structure is critical; ground vibrations will have their greatest effect on structures that vibrate harmonically in response to the vibrational frequency. Earthworks are more sensitive to low-frequency vibrations than to high-frequency vibrations. Because high-frequency vibrations occur in the near field (close to the earthquake center) and low-frequency vibrations occur in the far field, damage from seismic activity may be greater for facilities farther from the active fault than for facilities sited nearby.

Delineation between near- and far-field effects from seismic events, as well as the general propagation of seismic energy away from the source, depends upon:

- Source geometry (geology and structure)
- Earthquake intensity
- Destructive interference or magnification of seismic waves by the medium of transportation (the geologic environment).

The site geology is the critical factor influencing the potential impact of a seismic event on a facility. Seismic energies can be dampened or magnified and focused by different subsurface geological structures.

On May 2, 1983, a 6.5-magnitude earthquake occurred in Coalinga, California, with its epicenter approximately 5.5 miles to the southeast of a hazardous waste facility. Several hazardous waste surface impoundments were in operation at the facility at the time of the quake. Following the quake, the facility was inspected by an engineer for a report to the State of California Regional Water Quality Control Board. The following is an exerpt from his report:

> Although the facility suffered no significant damage, subtle evidences of strong ground shaking were observed across the site. A distinctive sign of severe ground shaking was the disruption of a thin surface veneer of dried soil. The drying of surface soil moisture from recent rains has resulted in the formation of an approximately 1/4-inch soil crust in many drainage swales. During seismic shaking, this crust had been "shattered" and underlying soil powder [has] been forced up through tiny cracks forming a distinctive soil pattern.
>
> Other observations of ground shaking included slumping of the surface of small stockpiles of sand, movement of loose soil clods on cut slopes, and small cracks in the surface layer of truck compacted roadways. Several soil slumps were observed along erosion cuts in an unused soil berm in the area of [a] future pond. This berm has no function in site design or operation. Wave action in containment ponds due to ground shaking was documented by a fresh oily residue approximately 6 to 8 inches above the static fluid level. Severe ground shaking also resulted in the displacement of two small gasoline storage tanks from their elevated mounting stands.
>
> Despite these signs of significant ground shaking, all perimeter waste containment and drainage control facilities were found free of any evidence of damage or signs of slope instability. All structures were carefully examined for the presence of tension cracks, soil sluffing, or unusual soil moisture or seeps. A system was immediately installed to monitor any unusual fluctuations of fluid levels in waste ponds and downgradient monitoring wells.
>
> The only observation of seismic impacts on waste impoundments was confined to the presence of tension cracks in (1) perimeter embankments of two interior oil recovery ponds that had been scheduled for reconstruction, and (2) an internal levee between [two] ponds. Both facilities located behind perimeter drainage control structures were immediately taken out of service and the fluids pumped to other containment areas.

> No significant damage was noted to any on- or off-site access roads and the site remained accessible throughout the period following the earthquake.

Another site, a landfill 5 miles from the quake, showed extensive cracking of embankments and fill material. Portions of the tops of an embankment slope had been displaced about 1 foot. However, no release of material resulted from this damage.

At two other sites, 25 and 100 miles from the epicenter, no signs of damage or shifting were observed during similar inspections.

6.9 SCOURING

Scouring is the erosion of the liner or sidewalls of a containment structure by the force of moving water. As a failure mechanism, scouring is of concern for surface impoundments, such as wastewater treatment lagoons, which use mechanical aerators as part of the water treatment process. Scouring can also occur due to wave erosion on the sides of a lagoon due to wind or from improper waste placement techniques. Scouring can cause failure of the liner with subsequent soil contamination and perhaps groundwater contamination.

Covering susceptible areas in the impoundment with gravel or rock (rip-rap) is one means to prevent scouring. Johnson and Cole (1976) reported that a bentonite liner in a paper-mill lagoon was protected from scouring by such covering. First the liner was covered with 3 to 4 inches of till, and then a 6-inch gravel cover was placed over the till in the most susceptible areas (e.g., at the base of the aerators). Sidewalls with 3:1 slopes were protected from scour by 6 inches of crushed stone that was covered with rip-rap. No problems have resulted in this facility. Johnson and Geisel (1979) reported that a clay liner used in a municipal wastewater treatment lagoon was protected by covering with gravel.

Voigts and Savage (1974) described a wastewater treatment lagoon lined with natural Michigan clay. Rip-rap was added to the slopes to protect against wave action and erosion, and a concrete scour pad was placed under the aerators to prevent scouring the liner. Stone was added around the scour pad out to 35 feet as an additional safety factor to prevent eroding the clay liner. The aerator supports, which penetrated the clay liner, were sealed with neoprene water stops in the middle of the compacted clay.

A failure due to scouring has occurred at a clay-lined sludge impoundment. The facility was originally designed to contain solid wastes. However, when put into service, the facility was instead used for the disposal of a hazardous sludge containing approximately 70 percent moisture. The sludge placement technique involved emptying the contents of a tank truck through a single pipe and allowing the sludge to drop down directly onto the liner. The force created by the sludge falling onto the liner has scoured or eroded away a section of the 3-foot clay liner.

6.10 FAILURES FROM DESIGN OR CONSTRUCTION ERRORS

The discussion of failure mechanisms has included the importance of design and construction techniques to prevent or minimize failures. Obviously, a design or construction error can contribute to the occurrence of a failure and in this sense could be considered a cause of failures. For example, an error in the slope stability analysis in the design stage could lead to construction of an unstable side slope. Construction problems are addressed briefly here; a more complete discussion of construction may be found in Chapter 5 of this document.

Daniel (1983) presented several conclusions based on case studies of leaking clay pond liners. He concluded:

- Desiccation of the clay liner between the completion of construction and the commencement of operations allows the clay to crack.
- A nonuniform moisture distribution in the soil results in clods with wet surfaces and dry, cracked interiors. This problem is caused by inadequate breakup of large clods prior to compaction, uneven water distribution by water trucks, and inadequate time for water to penetrate the soil.
- High permeabilities result from inadequate control of moisture, density, and compactive effort during liner compaction.
- Improper screening of incoming liner material can result in small roots, rocks, and lenses, and in other heterogeneities in the clay liner that can increase field permeability.

Examples of field installation problems can be found in the case study data presented in Chapter 7. At Site B, the depth of liquid waste was as great as 40 feet (bathtub) effect. This liquid entered an improperly sealed sand seam on the upper portion of the facility sidewall and contaminated an aquifer. At Site O, water eroded the area around a standpipe because a poor seal was obtained during installation. At a site not included in Chapter 7, excavation to depths that left an insufficiently thick layer of natural clay is suspected to have resulted in leakage problems. Sites where a steep slope has failed and required regrading and reconstruction are not uncommon.

Montague (1982) described a landfill in New Jersey that was constructed with an 18-inch primary clay layer and a 12-inch secondary clay layer with a leak detector layer between the two. The site was designed to receive chemical solids and dry chemical sludges from the manufacturer of organic chemicals. Seven months after operations at the site commenced, liquid was discovered flowing into the leak detector area between the two layers. The company operating the site stated there were two possible causes: (1) improper placement of the clay liner in the trench where the monitoring manhole pipe was laid, or (2) a break in the clay caused by truck traffic in the solid waste area. Attempts to locate the leak have been unsuccessful over the past 2 years. The leak indicates a failure in the primary liner, but there is no evidence of leakage through the secondary liner or of groundwater contamination (Montague, 1982).

Potential installation problems underscore the need for strict quality control and quality assurance in the design, construction, and operation of hazardous waste disposal facilities. The operator must be assured that the facility is constructed as designed with properties (as measured in the field or from representative samples) that are uniform and within specifications. A discussion of construction quality control and quality assurance may be found in Chapter 5 of this document.

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CHAPTER 7

CLAY LINER PERFORMANCE

7.1 INTRODUCTION

Knowledge of the performance of existing liner designs under field conditions is an important factor for evaluating the adequacy and practicality of various designs for compacted clay liners. As part of this project, an attempt was made to gather clay liner field performance information. At the onset of this effort, only clay-lined hazardous waste sites with lysimeters or leak detection systems were deemed suitable for our purposes. As work progressed, however, it became apparent that additional sites without lysimeters or leak detection systems that contained predominantly nonhazardous wastes might also be useful. In addition, several facilities that are unlined but located in deposits of low-permeability soil were included in the case study section. The inclusion of such sites is generally for the illustration of a specific performance problem.

The success or failure of a disposal site depends upon many factors, most of which are unrelated to the type of waste contained within. Hazardous and nonhazardous waste sites are probably equally susceptible to the physical failure mechanisms discussed in previous chapters. This is not the case, however, where chemical and clay interactions are important; in these cases, industrial and chemical waste disposal sites have a greater probability of displaying these effects than do municipal or nonchemical waste sites. Therefore, by including as many different types of sites (municipal, industrial, and chemical) as possible, problems resulting from most types of failure mechanisms can be "tested for" through the analysis of the site data.

This chapter discusses the various factors affecting clay liner performance as well as the techniques and problems of clay liner performance monitoring. Section 7.2 of this chapter is a detailed discussion of 17 facility case studies. This section includes such information as the age of the facility, liner specifications, construction techniques, waste types, local hydrology and geology, and facility performance. Section 7.3 contains a summary of the information presented in the case study discussions. The relationship between the number of successes and failures of the three facility types (lined with compacted soil, lined with admixed materials, or unlined) is also presented. Sections 7.4 to 7.7 include discussions on various factors that influence a facility's performance and methods for monitoring that performance. The final section, Section 7.8, presents the conclusions that can be drawn from the chapter.

7.2 CASE STUDIES

This section contains descriptions of 17 clay-lined facilities listed in Table 7-1. The data for these facilities were obtained from State and Federal agencies, commercial waste disposers, clay liner design and construction firms, and the industrial sector. Information on approximately 50 other disposal facilities was also obtained during this effort and previous data-gathering efforts. In most cases, either necessary information was not available or the facility did not have any unique features worthy of discussion.

7.2.1 Criteria for Site Selection

Several important sets of data are needed in order to evaluate the effectiveness of a disposal site. These include:

- Geologic and hydrogeologic site characteristics
- Types of waste in the site
- Geotechnical characterization of the clay
- Leachate collection, leak detection, or groundwater monitoring data
- Physical description of the site
- Liner description
- Construction procedures.

Many times, while the files on specific sites were reviewed, it became obvious that critical sets of data were either fragmentary or not available. One reason for this is that many sites had been "grandfathered in" when the States started permitting disposal facilities. In this circumstance, important information, which is now required before a site can receive a permit, was never developed or submitted to the States when these older sites were started. For this reason, even though we know about the existence of many old clay-lined sites, their incomplete documentation makes them unsuitable for our purposes.

In some cases, when sites have been suspected of causing groundwater pollution, the State or landfill operator has initiated a hydrogeological study designed to evaluate the performance of the clay or soil leachate barrier. Sites where these studies have been performed are among the best documented and are included in our data base. However, the motivation for performing these studies was the suspicion of poor performance, and their inclusion here may distort our perception of the capabilities of clay liners in general. We have no way of including the hundreds of undocumented clay-lined sites that may be performing well.

TABLE 7-1. CLAY-LINED FACILITY INFORMATION

Site	Startup date	Waste type	Liner description (liner components listed from the top down)	Facility performance and comments
A	1976	NH[a] and HW[b]	-Unrecompacted in situ glacial till.	Original monitoring well installation problem has been corrected. No recent performance problems have been reported.
B	1971	NH and HW (including unsolidified liquids)	-Zone-of-saturation landfill. -In situ glacial till. Sand or gravel seams were excavated and then backfilled with clay. -Leachate collection system.	Bathtub effect has caused inward hydraulic gradient to reverse. Severe groundwater contamination has resulted.
C	1977	HW (including organic solvents, heavy metal sludges and acids) high-density polyethylene	-In situ clay-shale soil containing calcium carbonate nodules and seams. Sand or gravel seams were excavated and then backfilled with soil.	Groundwater contamination has occurred, possibly due to a reaction between the low pH waste and the calcium carbonate inclusions in the local soil.
D	1979	HW	-Leachate collection system. -80-mil (HDPE) liner. -5-foot recompacted clay liner.	No performance problems.
E	Late 1970's	HW	-French drain above liner in each cell. -Recompacted Demopolis Chalk.	Minor problems, including buildup of leachate head, have been corrected.
F	1976	HW	-Leachate collection system -Flexible membrane liner (FML) (Hypalon® or HDPE). -10-foot recompacted clay liner from borrow site.	Leachate removed from collection system is less contaminated than the groundwater. Contamination was caused by an adjacent facility requiring remedial action.
G	1955 Leachate collection system added in 1982.	HW	-Leachate collection system. -Unrecompacted glacial till.	Contamination in well is thought to have come from an adjacent abandoned drum storage facility by way of a sand and gravel seam.
H	1980	75% MSW[c] 25% paper mill sludge	-Leachate collection system. -4-foot recompacted clay liner. -Three lysimeters	Initial collection of liquid in lysimeters thought to be soil moisture release following construction
I	1980	Liquid HW	-5-foot recompacted clay liner. -Leak detection system. -5-foot recompacted clay liner.	Desiccation cracks formed in unprotected liner prior to filling with waste. When pond was filled, waste migrated into the detection system.
J	1979	HW (liquids and solids)	-5-foot recompacted clay liner. -Leak detection system. -1-foot recompacted clay liner.	Chlorinated organic liquid placed in pond caused liner failure

(continued)

TABLE 7-1 (continued)

Site	Startup date	Waste type	Liner description (liner components listed from the top down)	Facility performance and comments
K	1981	Liquid HW	-3-foot recompacted clay liner. -Leak detection system. -1-foot recompacted clay liner.	Liquid volumes collected in detection system were used to calculate liner permeability. Values ranged from 4×10^{-8} to 3×10^{7} cm/s. Recent major earthquake 100 miles north of the facility caused no damage.
L	1978	HW	-Leachate collection system. -Approximately 2-foot recompacted clay liner. -Leak detection system. -1-foot recompacted clay liner. -Approximately 1-foot compacted soil liner. -Geotextile.	Failure in upper liner has occurred. Lower liner is still functioning.
M	1977	HW	-Leachate collection system. -1-foot recompacted borrow clay soil liner. -Leak detection system.	No performance problems.
N	1974	MSW	-Leachate collection system. -4-inch bentonite (averaging 11 percent by weight) and sand liner. -Two lysimeters.	Groundwater contamination has been detected at several monitoring points.
O	1980	HW	-Leachate collection system. -4-foot-thick recompacted local clay liner with 3 percent bentonite and 3 percent lime added.	Cap failure caused increased leachate volumes. Problem was corrected by replacing section of cap.
P	1980	HW	-Leachate collection system. -5-foot recompacted soil liner including a 1-foot layer of bentonite (9 to 12 percent) and soil. -Leak detection system. -6-inch bentonite and soil layer. -5-foot in situ soil layer.	Small amounts of liquid have been collected in the leak detection system. Monitoring wells have shown no significant changes.
Q	1976	HW	-Leachate collection system. -4-inch bentonite/soil liner. -Leak detection system. -4-inch bentonite/soil liner.	Liquid volumes collected in detection system were used to calculate the liner permeability. Values ranged from 3×10^{-8} to 6.5×10^{-8} cm/s.

[a] NH - Nonhazardous

[b] HW - Hazardous waste.

[c] MSW - Municipal solid waste

As one might suspect, the best documented sites are those relatively few that have been built after the institution of strict permitting requirements at the State and Federal level. In general, permit files contain hydrogeological reports and engineering reports that document the installation of the facility and the characteristics of the clays used. Typically, however, these sites are only a few years old and therefore do not supply much data relevant to the prediction of the long-term performance of clay liners.

The site data presented in the following sections include all of the relevant information that was made available to the authors for each site. As discussed in the previous paragraphs, the quality, quantity, and type of these data were quite varied.

7.2.2 Site A

7.2.2.1 Physical Description--

This sanitary and hazardous waste landfill is located in an area of glacial till soils. The liner at this facility consists of the unrecompacted glacial till, which has a relatively low permeability. A plan view of the facility is presented in Figure 7-1.

7.2.2.2 Startup Date--

This facility began operations in 1976.

7.2.2.3 Local Geology and Hydrology--

This facility is located in the midwestern United States. Average annual precipitation at this site is 35 inches.

The soil in the vicinity of the landfill is described as a clay-loam till. Soil borings indicate that it extends to depths of 150 feet. Underlying the till is a thick formation consisting of limestone, dolomite, sandstone, and slate. Occasional gravel, silt, or sand lenses are present in the upper till layers. It is not known if these areas were excavated and backfilled with clay prior to waste placement.

There is very little information about the groundwater levels in this area. However, it is known that the groundwater flows in a northeasterly direction and that the facility is located in an area where the possibility of groundwater contamination is low.

Monitoring wells are located both upgradient and downgradient of the site. Samples are analyzed quarterly for abnormalities in groundwater conditions.

7.2.2.4 Waste Type--

Both sanitary and hazardous wastes (including drummed combustibles and leachable metals) are accepted at this facility.

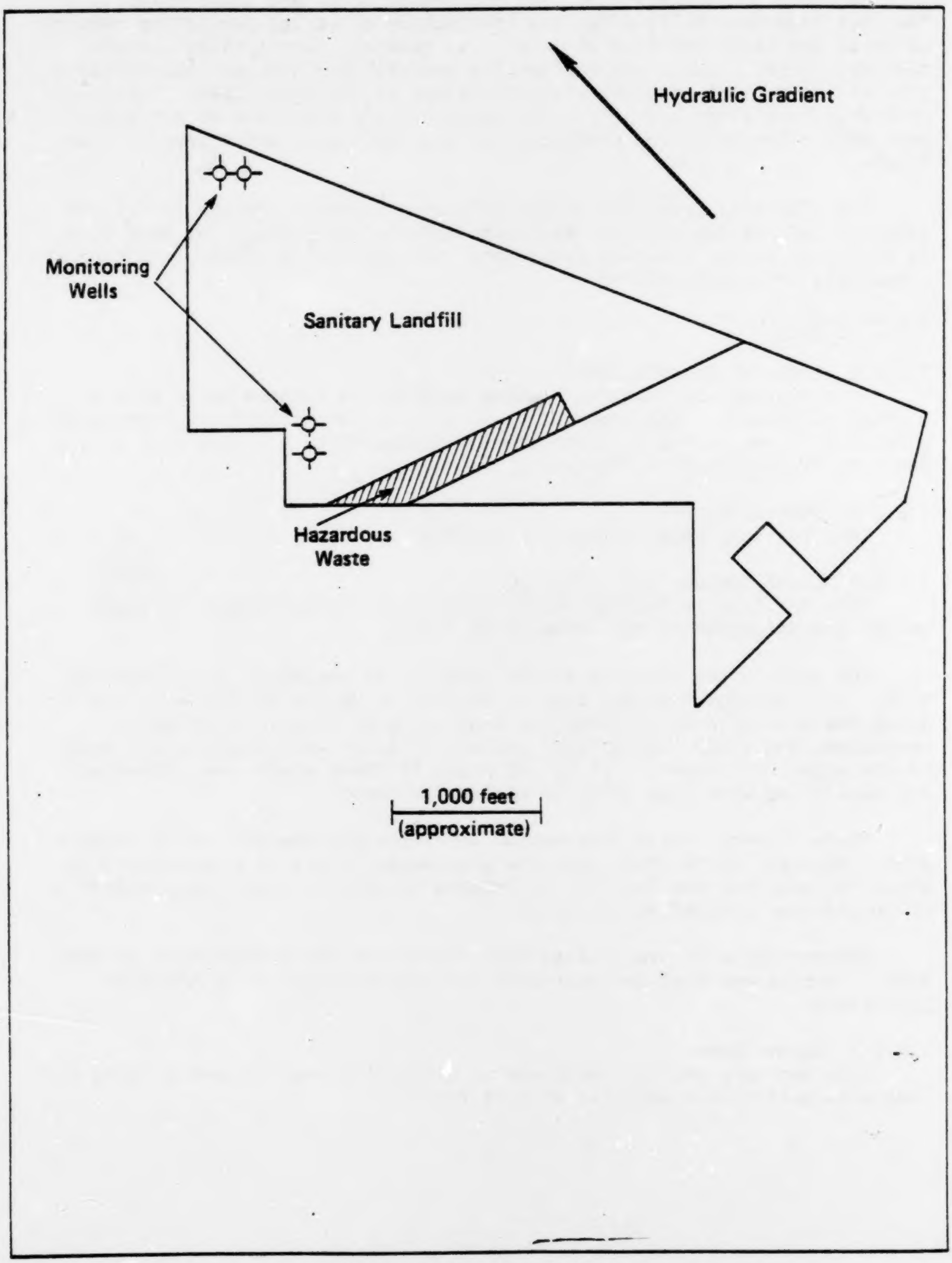

Figure 7-1. Plan view of site A.

7.2.2.5 Liner Description--

The liner at this facility consists of unrecompacted in-situ glacial till. This material has the following characteristics.

• Permeability	3.0×10^{-6} to 5×10^{-8} cm/s (triaxial tests with leachate and water)
• Liquid limit	22 - 26%
• Plasticity index	2 - 10%
• Moisture content	12 - 22%
• Amount passing No. 200 sieve	75 - 95%.

Compatibility testing performed by a private firm has indicated that the leachate will not react with the surrounding liner soils in a way that would significantly increase its hydraulic conductivity. The major organic components of the test leachate were phenol, methylene chloride, 1,1,1-trichloroethane, toluene, 1,1-dichloroethane, and diethylphthalate.

7.2.2.6 Liner System Installation--

This is an unlined facility. Methods used for the excavation were not available.

7.2.2.7 Performance--

Analysis of groundwater samples taken from the present monitoring wells has not revealed any leachate migration problems. However, the original monitoring wells were improperly installed, having contained steel mill slag where clean gravel should have been used. Samples taken from these wells in 1976 had pH levels between 10.7 and 11.3. The analysis also revealed a chemical oxygen demand (COD) of 3,193 ppm and a chloride level of 1,028 ppm. These elevated levels were attributed to the steel-mill slag in the well casings. Samples taken from newly constructed wells in 1981 had pH levels of around 7, COD levels ranging from less than 10 ppm to approximately 60 ppm, and chloride levels ranging between 4 ppm and 12 ppm.

7.2.3 Site B

7.2.3.1 Physical Description--

This 166-acre facility contains an 80-acre zone-of-saturation, or intergradient, landfill. A small section of the landfill has a recompacted clay liner; the remainder of the landfill is unlined. A leachate collection system (LCS) lies between the waste and the landfill bottom. A plan view of the facility is presented in Figure 7-2.

7.2.3.2 Startup Date--

This landfill began accepting waste in 1971.

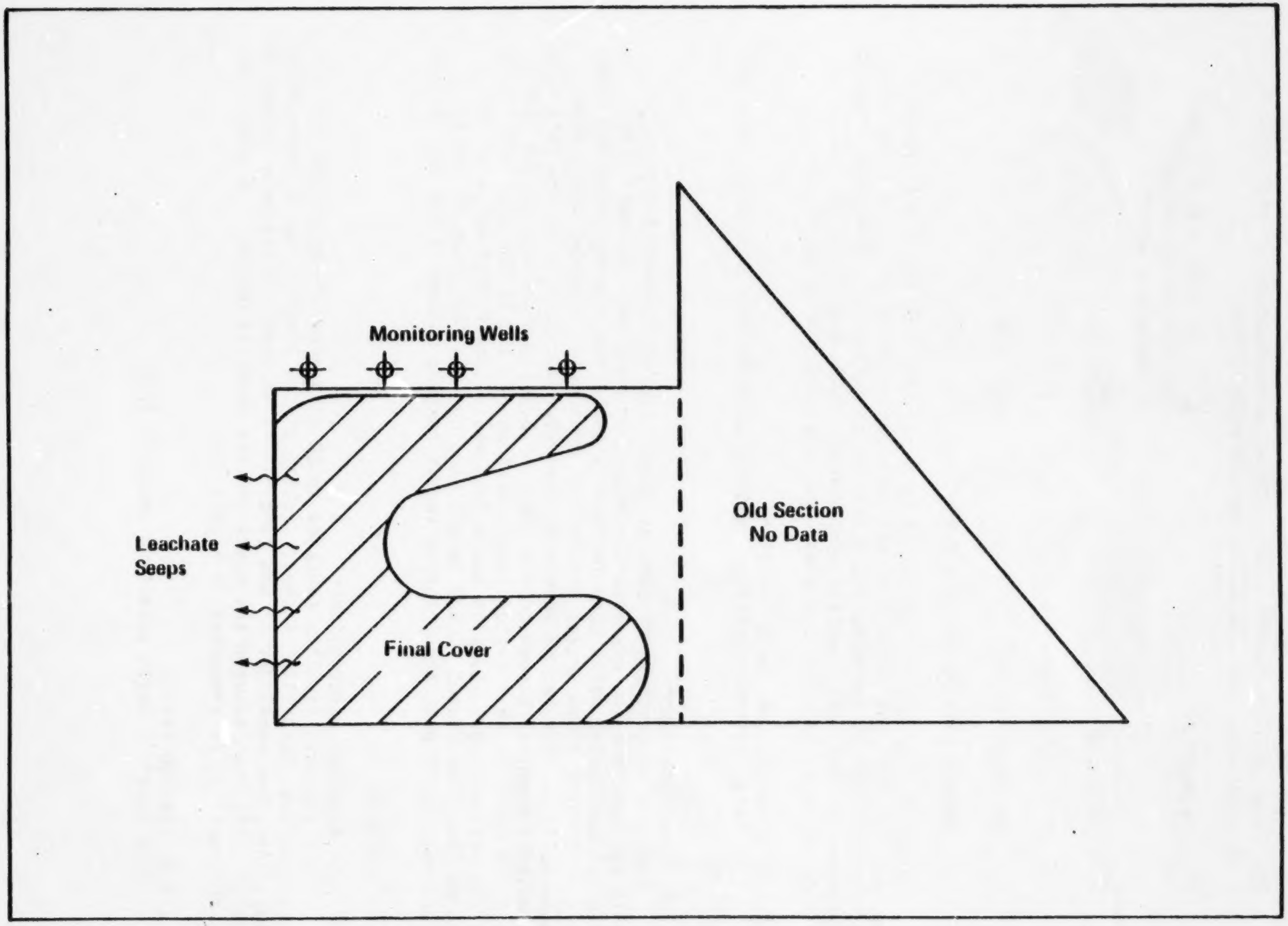

Figure 7-2. Plan view of site B.

7.2.3.3 Local Geology and Hydrology--

This facility is located in the northern central portion of the United States. Average annual precipitation at this facility is approximately 30 inches.

The site is located in a glacial till, which consists primarily of clay and silt. Site investigations have shown that sand and gravel lenses are present throughout the till. Underlying the glacial till is a layer of moderately fractured dolomite up to 200 feet thick. A permeable aquifer exists in this bedrock layer. The glacial till is saturated to within 10 feet of the ground surface. Groundwater flow was in a southeasterly direction prior to the development of the disposal facility.

7.2.3.4 Waste Type--

This facility contains municipal solid waste, nonhazardous industrial waste, and hazardous waste. Included in this are large amounts of plating sludge and pickle liquor as well as smaller amounts of various solvents. Liquid wastes were not solidified prior to their disposal.

7.2.3.5 Liner Description--

The liner at this facility is a minimum of 30 feet of in situ glacial till. The glacial till has the following characteristics:

Soil Characteristics	Average Value
• Amount passing No. 200 sieve	68%
• Clay	40%
• Liquid limit	29%
• Plastic index	14%
• Field permeability	7.5×10^{-6} cm/s
• Laboratory permeability	9×10^{-8} cm/s.

Lenses of sand or gravel that were encountered during site construction were excavated and backfilled with a minimum of 5 feet of recompacted clay. A leachate collection system is located on top of the landfill base. Specifications for this system were not available.

7.2.3.6 Liner System Installation--

No information on the excavation and construction of this facility was available.

7.2.3.7 Performance--

Recently, three major and numerous minor leachate leaks have been observed in the western side of the final cover (see Figure 7-2). The State geologists attribute these leaks to the 26-foot hydraulic head within the landfill. One geologist estimated that, at times, the head is as great as 40 feet. This high hydraulic head has caused the inward gradient of this zone-of-saturation landfill to reverse.

A second problem encountered at this site is the migration of leachate into the groundwater in a very small area on the northern edge of the landfill (see Figure 7-2). Excavations in the problem area revealed that in one spot a permeable deposit that had been improperly sealed with clay was below the level of the waste material. Leachate entered this permeable layer and was detected in a monitoring well. Remedial action consisting of the installation of a clay cutoff wall and leachate removal has resulted in some improvement in the groundwater quality; however, significant contamination is still present.

7.2.4 Site C

7.2.4.1 Physical Description--

The facility consists of 31 small drum disposal trenches, 4 treatment ponds, and 2 evaporation ponds (see Figure 7-3). The total storage and disposal area is approximately 16.5 acres.

The ponds and trenches were excavated in the local natural soil. If sand seams or other potentially troublesome areas were located in the pond bottoms, these areas were recompacted. No recompacting was done in the disposal trenches. Several monitoring wells surround the facility.

7.2.4.2 Startup Date--

The facility was opened in 1977, but because of groundwater contamination it was forced to close in January 1982. Since that time, changes have been made and parts of the facility are again in operation.

7.2.4.3 Local Geology and Hydrology--

This facility is located in the midwestern plains of the United States. Average annual precipitation in this area is approximately 25 inches.

Investigations have indicated the presence of two water-bearing zones beneath the facility at depths of approximately 40 and 50 feet below the ground surface. Several springs and a creek are to the north of the facility. The relationship between the groundwater zones and springs is unknown.

Soil borings indicate that approximately 3 feet of topsoil overlie 30 to 40 feet of weathered clay-shale. This weathered clay-shale is a plastic silty to highly plastic clay (CL to CH) with calcium carbonate nodules and seams throughout. Bedrock lies under the clay-shale layer. An example of a typical soil boring is illustrated in Figure 7-4.

7.2.4.4 Waste Type--

Various hazardous wastes including organic solvents, heavy metal sludges, and acids were held in the treatment and evaporation ponds and disposed of in the drum storage area.

7.2.4.5 Liner Description--

While this facility is almost completely unlined, potential problem areas uncovered by the pond excavations, such as sand seams, were recompacted.

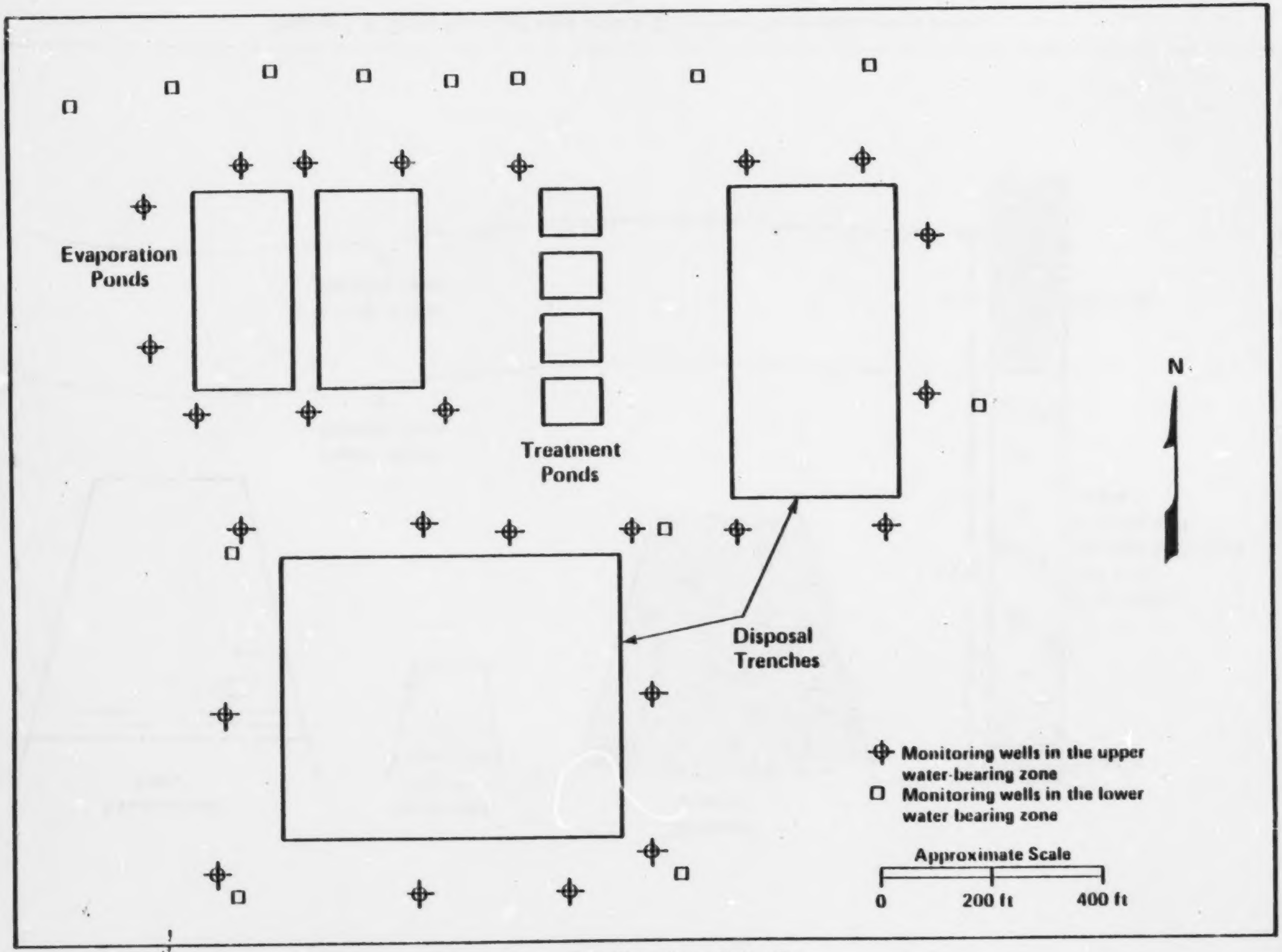
Evaporation Ponds
Treatment Ponds
Disposal Trenches
N
Monitoring wells in the upper water-bearing zone
Monitoring wells in the lower water bearing zone
Approximate Scale
0
200 ft
400 ft

Figure 7-3. Plan view of site C.

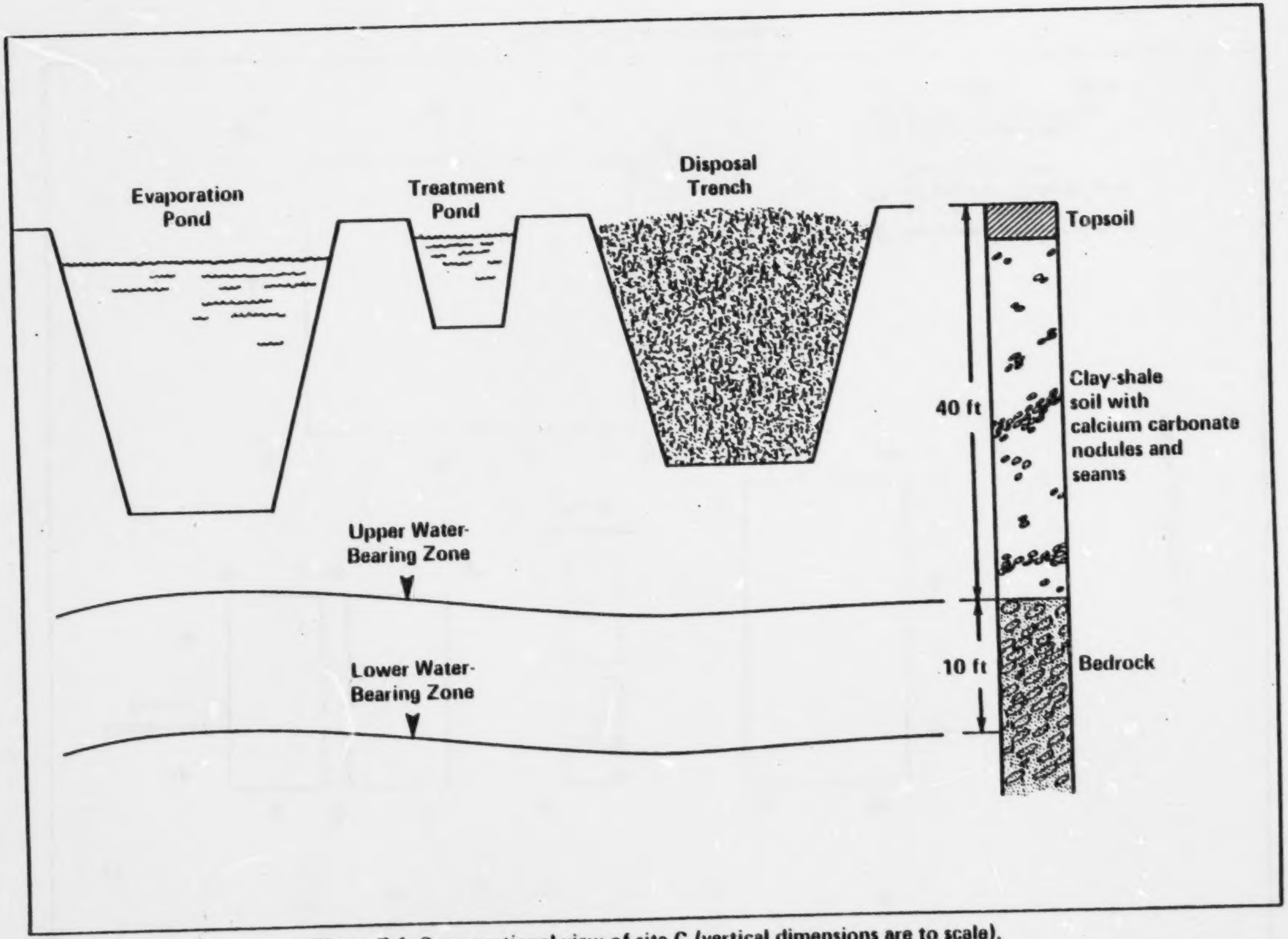

Figure 7-4. Cross-sectional view of site C (vertical dimensions are to scale).

The local clay-shale soil was the only barrier to waste migration. A cross section of the facility is presented in Figure 7-4.

7.2.4.6 Liner System Installation--

Information about the procedures used to excavate and recompact the disposal areas was not available. Methods used to install the monitoring wells were also not available.

7.2.4.7 Performance--

Analysis of groundwater samples indicates that contamination has occurred. Elevated concentrations of chromium, arsenic, barium, cadmium, lead, and mercury were detected near the disposal ponds and trenches. However, except for chromium, none of the heavy metals were detected in offsite monitoring wells. In addition to heavy metal contamination, several volatile organics such as chloroform, trichloroethane, dichloroethane, benzene, and methylene chloride were detected in both onsite and offsite monitoring wells. The highest levels of volatile organics were found onsite in the uppermost water-bearing zone in wells closest to the disposal areas. Base-neutral-extractable organics and acid-extractable organics were also detected in onsite and some offsite wells (see Table 7-2).

Investigations of the groundwater contamination at this facility suggest that the treatment ponds are the major source of contamination in the uppermost water-bearing zone. This conclusion is based on the fact that the materials in the ponds were usually of low pH and would be reactive with the carbonate seams and inclusions in the surrounding soil. Several of the trenches were used for the disposal of highly acidic waste oil reprocessing sludges. These sludges may have also attacked the surrounding materials and allowed contamination to occur.

7.2.5 Site D

7.2.5.1 Physical Description--

This recompacted clay- and synthetic-lined hazardous waste disposal facility consists of two large disposal sections, each of which is divided into several smaller disposal cells. These in turn are divided into three subcells each for the segregation of different waste types. Four of the five disposal cells in Section I are closed, and the remaining cell is currently active. The first of 10 proposed disposal cells in Section II is under construction. This construction is scheduled to be completed in the fall of 1984, at which time landfilling operations will be initiated. A plan view of the facility is presented in Figure 7-5.

7.2.5.2 Startup Date--

Construction of this facility was started in 1979.

7.2.5.3 Local Geology and Hydrology--

This facility is located in the southeastern United States. Average annual precipitation at this site is approximately 47 inches.

TABLE 7-2. GENERAL OCCURRENCE OF CHEMICAL PARAMETERS IN THE GROUNDWATER AT SITE C

Parameter tested for	No. parameters tested for	No. parameters detected and highest concentration (ppb)			
		Onsite		Offsite	
		Upper water-bearing zone	Lower water-bearing zone	Upper water-bearing zone	Lower water-bearing zone
Volatile organics-					
No. found	31	20	19	18	6
Highest concentration		370,000	3,100	79,000	510
Trace metals-					
No. found	8	8	8	8	7
Highest concentration		20,000	500	300	500
Acid-extractable organics					
No. found	11	6	ND	3	ND
Highest concentration		24,000	ND	3,300	ND
Base-neutral-extractable organics					
No. found	46	11	3	6	1
Highest concentration		410	83	250	160

ND = No parameters of this type were detected.

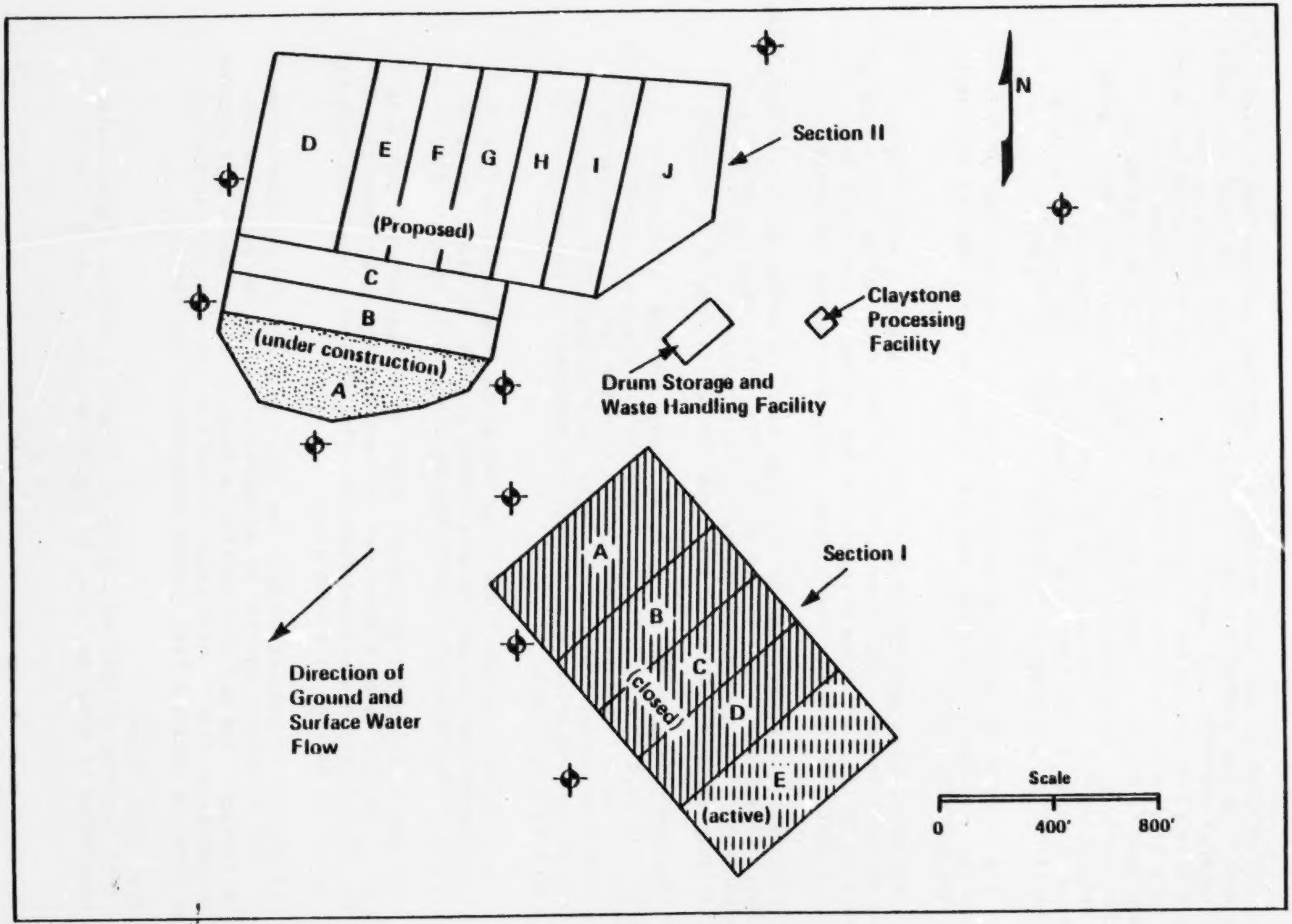

Figure 7-5. Plan view of site D.

Information on the local geology and hydrology was obtained through the use of 32 exploratory borings. These borings have confirmed that there are several distinct soil layers. The lowest of these layers is the Tuscaloosa Formation, which is overlain by the Black Mingo Formation, which in turn is overlain by Quaternary deposits. The upper surface of the Tuscaloosa Formation is located approximately 55 to 120 feet below the ground surface. This material consists of clayey sands and clays. Lenses or beds of coarse to fine sand are located in this formation. In situ hydraulic conductivity testing of this formation revealed that it has a permeability in the range of 1.09×10^{-6} to 4.01×10^{-6} cm/s.

The Black Mingo Formation encountered at this site consists of four subunits: the Basal Clay Unit, the Opal Claystone Unit, the Red Sand Unit, and the Buhrstone Unit.

The Basal Clay Unit consists of very hard black clay with frequent sand and silt lenses. The upper surface of this material is located approximately 60 to 115 feet below ground level and varies in thickness from 1.5 to 15.5 feet. In a few of the borings, this material was not present.

The Opal Claystone Unit, on the other hand, is either above the Tuscaloosa or Basal Clay Formations over the entire site. This material consists of low-permeability (exact value unknown) silt and clay with very few sand inclusions. The thickness of this formation varies from 20 to 60 feet.

The Red Sand Unit, which is located above the Opal Claystone Unit, consists of sand to clayey silts. The silts and clays are generally present as 1/4-inch to 2-inch layers interbedded with the sand. The thickness of this formation varies from 0 to 15 feet. In situ hydraulic conductivity testing of this material revealed that its permeability ranges from 6.24×10^{-5} to 6.43×10^{-4} cm/s.

The final unit of the Black Mingo Formation, the Buhrstone Unit, is found scattered over the entire area. When present, it ranges up to 14 feet thick. This material is described as clayey silt to silty clay.

The Quaternary deposits contain layers of variegated clay that were originally thought to be a source of low-permeability liner material. Further investigations revealed that the silt and sand content was much too high for it to be used as a base liner.

An aquifer is present in both the Tuscaloosa and the Red Sand Formations. The Tuscaloosa aquifer is approximately 60 to 70 feet below the ground surface. The Red Sand aquifer is roughly 40 feet above this aquifer. The direction of flow in both cases, as well as the direction of surface-water flow, is toward a lake located southwest of the facility.

7.2.5.4 Waste Type--

This facility accepts most types of hazardous wastes. These wastes are classified as alkaline, acid, or organic and are disposed of with similar wastes only. This is accomplished through the use of three subcells that are incorporated into all of the cells in Section II of the facility. The size of each of the subcells was determined so that maximum separation of potentially incompatible wastes was provided and based on past operating experience.

7.2.5.5 Liner Description--

The liner at this facility consists of 5 feet of recompacted clay (maximum permeability of 1 x 10^{-7} cm/s) brought to the facility from a borrow pile located several miles away. Ten-foot sidewall liners are present above the top of the Opal Claystone Unit (see Figure 7-6). An 80-mil high density linear polyethylene (HDPE) membrane liner lies on top of the recompacted clay. This liner is protected with a soil layer that is 9 inches deep on the bottom and 24 inches deep on the sidewalls. A leachate collection system is located on top of the protective soil layer (see Figure 7-6).

The subcell separation berms are constructed of recompacted clay and are built up in 5.5-foot-high sections as the landfill operations progress. The side slopes of these berms are 1:1.

7.2.5.6 Liner System Installation--

Prior to excavation of the landfill cells, a channel to drain the perched water table within the Red Sand Unit was constructed. After this channel was completed and in operation, excavation of the landfill cells was initiated. Side slopes were cut on a 3:1 (horizontal:verticle) slope down to the top of the Opal Claystone Unit. At this location, a 17-foot-wide "bench" with a 3-foot-wide trench was excavated into the top of the opal claystone (see Figure 7-6). This area was excavated to provide room for a thicker liner. Below the "bench," side slopes were also 3:1. A minimum of 10 feet of the opal claystone was left as foundation material. At the leachate collection system sump locations, approximately 4.5 feet of opal claystone remained as the foundation.

After the sides and base of a cell were graded, clay was brought to the site from the borrow area. Before the clay was placed in the excavation, a "test fill" was constructed with borrow soil that was selected based on the basis of laboratory screening procedures. The "test fill" was constructed in the field with equipment and procedures that were to be employed during liner construction. Undisturbed Shelby tube samples were taken from the completed "test fill" and were tested in the laboratory for density, moisture content, and permeability. The clay was compacted in 6- to 8-inch lifts with either a sheepsfoot or smooth-drum vibratory roller. The liner specifications required a permeability of 1 x 10^{-7} cm/s or less. Testing results from the construction quality assurance (CQA) tests on the installed liner were not available; however, results from tests performed on Shelby tube samples taken from the "test fill" were as follows:

•	Density	114.9 - 120.0 lb/ft^3
•	Water content	22.6 - 29.8%
•	Permeability	1.8 x 10^{-8} to 8.0 x 10^{-8} cm/s.

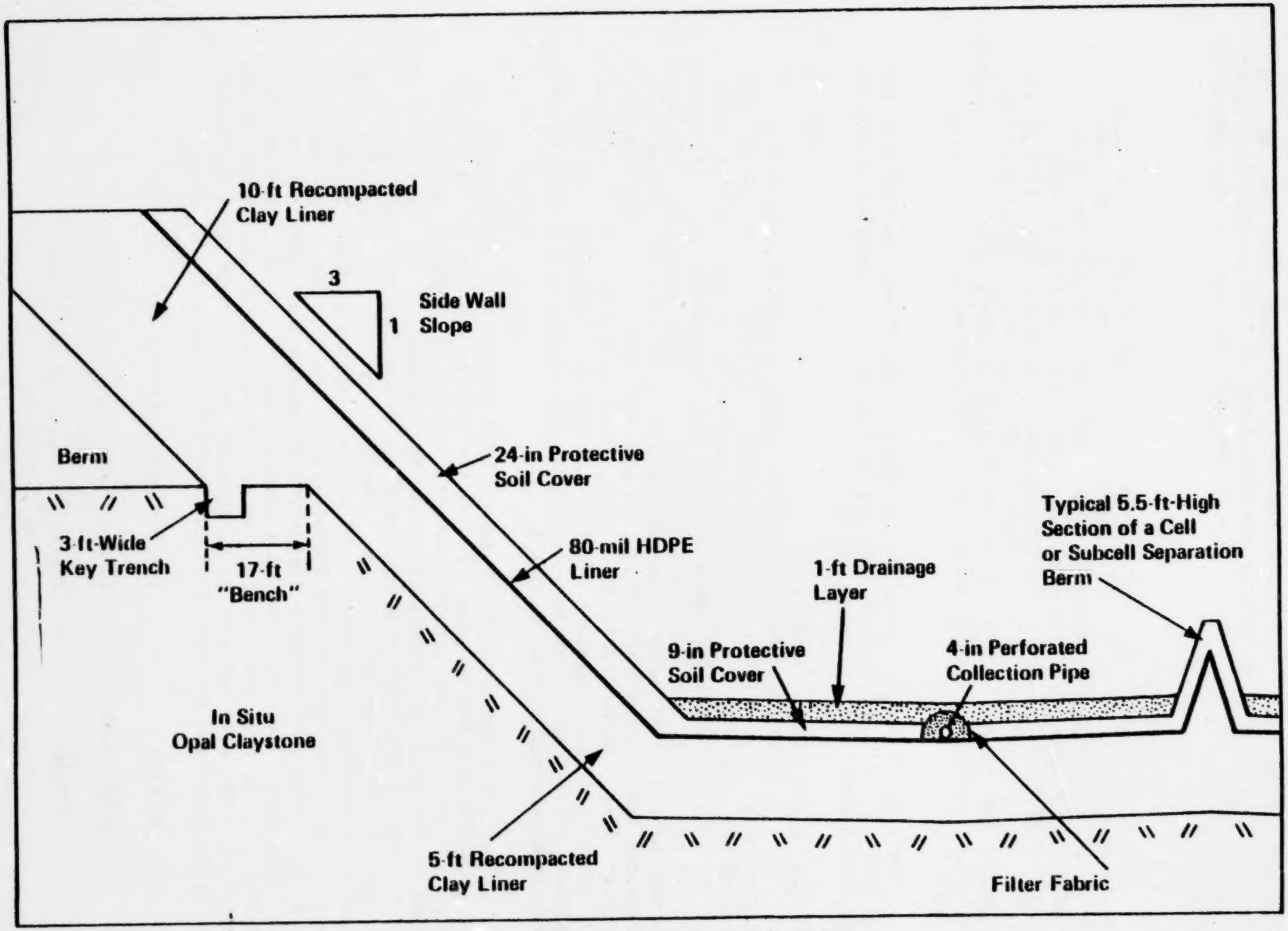
10-ft Recompacted Clay Liner
3
1
Side Wall Slope
Berm
24-in Protective Soil Cover
3-ft-Wide Key Trench
17-ft "Bench"
80-mil HDPE Liner
1-ft Drainage Layer
Typical 5.5-ft-High Section of a Cell or Subcell Separation Berm
9-in Protective Soil Cover
4-in Perforated Collection Pipe
In Situ Opal Claystone
5-ft Recompacted Clay Liner
Filter Fabric

Figure 7-6. Cross-sectional view of site D liner.

7.2.5.7 Performance--

Groundwater samples taken from upgradient and downgradient monitoring wells have not been statistically different since the start of waste placement.

7.2.6 Site E

7.2.6.1 Physical Description--

The facility presently occupies approximately 100 acres, with room for expansion to 340 acres. There are over 20 disposal trenches at various stages of operation as well as 4 active evaporation ponds (see Figure 7-7).

7.2.6.2 Startup Date--

This facility became active in the late 1970's.

7.2.6.3 Local Geology and Hydrology--

This facility is located in the southeastern United States. Average annual precipitation at this facility is 50 inches.

It is located in a natural clay formation called the Demopolis Chalk Formation. Testing of the Demopolis Chalk Formation has shown that it has the following properties:

- Average permeability 4.0×10^{-8} cm/s
- Permeability to polychlorinated biphenyl (PCB) liquids:
  - Light oils and PCB's, 9×10^{-9} cm/s
  - Heavy oils and PCB's, 1.5×10^{-10} cm/s
- 88% of the material passes the No. 200 sieve
- 35% of the material is less than 0.001 mm in diameter
- Liquid limit of 31%
- Plastic limit of 20%.

Soil borings in the area have shown that the clay is a minimum of 500 feet thick and in some places extends to depths of over 700 feet.

The closest usable groundwater is located below this clay formation. However, a few locations of perched water are in the vicinity of the site. In addition, the Demopolis Chalk Formation is saturated to within 10 to 30 feet of the ground surface.

The 100-year projected flood elevation for a creek, located 3,000 feet north of the facility, is 125 feet above mean sea level. This should present no problem to the facility because the disposal trenches are located

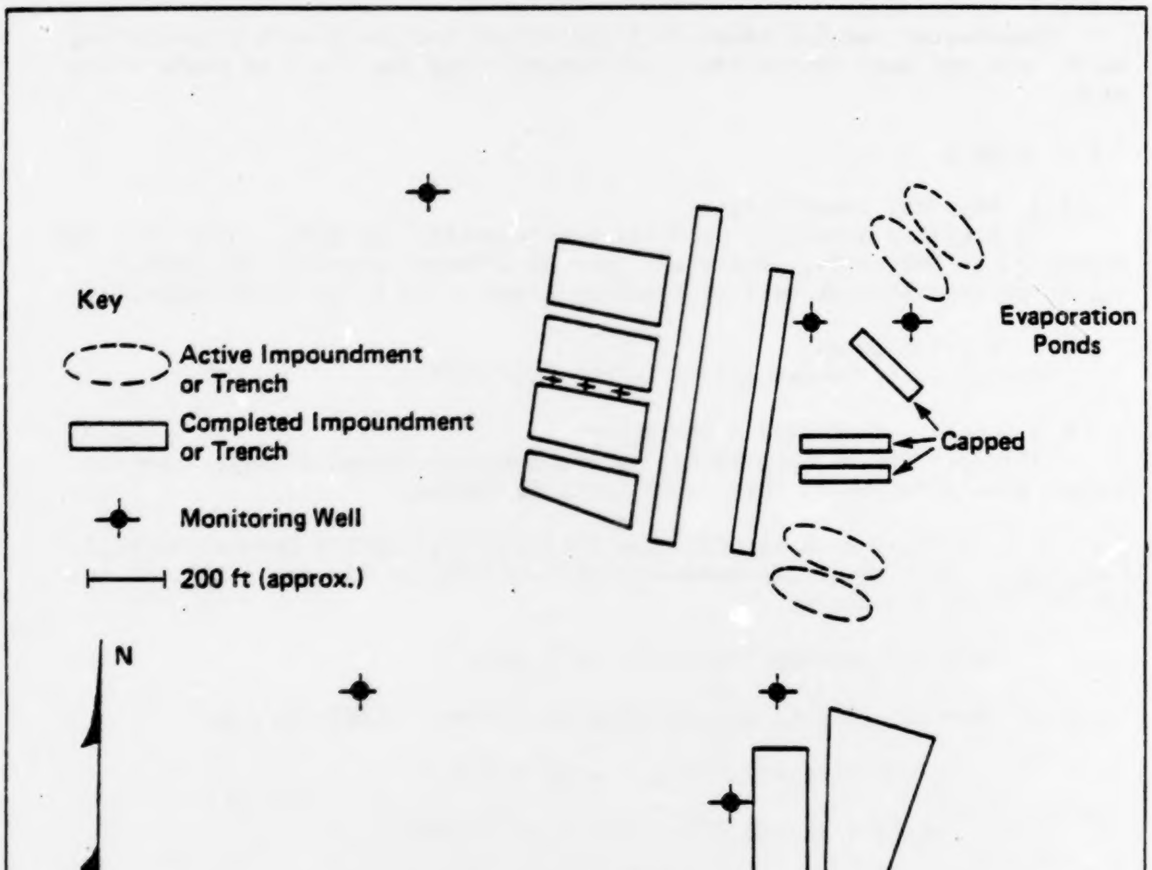

Figure 7-7. Plan view of site E.

at points that range from 180 to 220 feet above mean sea level. The lowest elevation of the site is 148 feet above mean sea level.

7.2.6.4 Waste Type--

The site is designed to be a full-service facility that accepts all types of hazardous wastes. The majority of the material received for disposal consists of acidic and caustic industrial wastes, heavy metal sludges, contaminated containers, sludge, and oil sludges. All wastes received by the facility are handled separately, and only compatible materials are placed within proximity of each other.

PCB wastes are handled separately and only PCB's and PCB-compatible wastes will be located in the same disposal trench.

7.2.6.5 Liner Description--

Each disposal trench is lined with recompacted native soil. Construction information concerning the number and thickness of lifts was not available. A calculated amount of absorptive material is placed on top of the recompacted bottom of each trench. The wastes are then placed on top of this material and covered with 6 to 8 inches of the natural clay. The final trench covers are also constructed with natural clay.

The current monitoring system consists of the following:

- A subsurface collection system and a secondary retention dam. This system would collect any materials that might migrate.
- Several monitoring wells placed within proximity to active trenches.
- Observation wells surrounding the entire facility.

A French drain system is located at the lowest point in each trench. In the event that liquid is collected in this drain, it would be pumped out and treated as incoming waste.

7.2.6.6 Liner Installation--

No information was available.

7.2.6.7 Performance--

In general, this facility is operating as planned. There have, however, been a few minor problems. Groundwater from a more permeable weathered section of the Demopolis Chalk Formation has infiltrated the sidewalls of trenches 15 and 19. At trench 15, this problem was remedied by installing a cutoff wall on the south-southeast side of the trench. Recompaction of the entire sidewall at trench 19 eliminated the groundwater flow into this trench. More recently, large volumes of leachate (up to 50 feet deep) have been accumulating in some of the trenches with interim final covers. These large volumes of leachate are attributed to the infiltration of groundwater and precipitation. Frequent pumping of the leachate has helped to reduce these volumes. Final covers consisting of 3 to 5 feet of recompacted Demopolis Chalk and a HDPE liner are scheduled to be installed by the end of 1986. In addition, the State monitoring well analysis has shown trace levels (92 ppb maximum) of 12 organic compounds in groundwater samples. An investigation of their source indicates that they may have come from the well casing materials.

7.2.7 Site F

7.2.7.1 Physical Description--

The 380-acre facility consists of five secure land disposal cells with a total area of 25 acres. Cells 1, 2, and 3 are closed; cell 4 is currently being filled; and cell 5 is under construction. The remainder of the facility is devoted to sanitary landfilling, waste treatment operations; and recovery of lime from the old slag disposal operation.

The five cells are lined with 10 feet of recompacted clay soils. On top of the clay liner is a flexible membrane liner (FML), which in turn is covered with a 1-foot layer of recompacted clay. A layer of geotextile lies above the 1-foot clay liner. Finally, a 12-inch drainage blanket containing slotted 4-inch corrugated HDPE pipe at a minimum of 100-foot intervals is installed on top of the geotextile (see Figure 7-8).

7.2.7.2 Startup Date--

The facility is located at the site of a slag disposal operation that has not been active for 90 years. The clay-lined facility as it currently exists has been in operation since 1976.

7.2.7.3 Local Geology and Hydrology--

This facility is located in the northeastern United States. Average annual precipitation in this area is approximately 36 inches.

Numerous onsite borings and test pits have revealed that the site is an old disposal area covered with waste industrial slag ranging in depth from approximately 7 feet to 45 or 50 feet. The disposed slag fill ranged in size from silt-sized particles to boulders. Beneath the slag was a layer of marsh silt that ranged in thickness from a few inches to as much as 5 feet.

Below the slag and marsh silt materials covering the entire site was a 6-foot layer of lacustrine clays. These clays contained lenses of gray nonplastic silts and clay with minor amounts of fine sand. The presence of these lenses throughout the lacustrine clay eliminated it from consideration as an in situ liner material. Laboratory analysis of the lacustrine clay indicated that it had the following properties:

• Natural water content	26.5 - 42.8%
• Liquid limit	50%

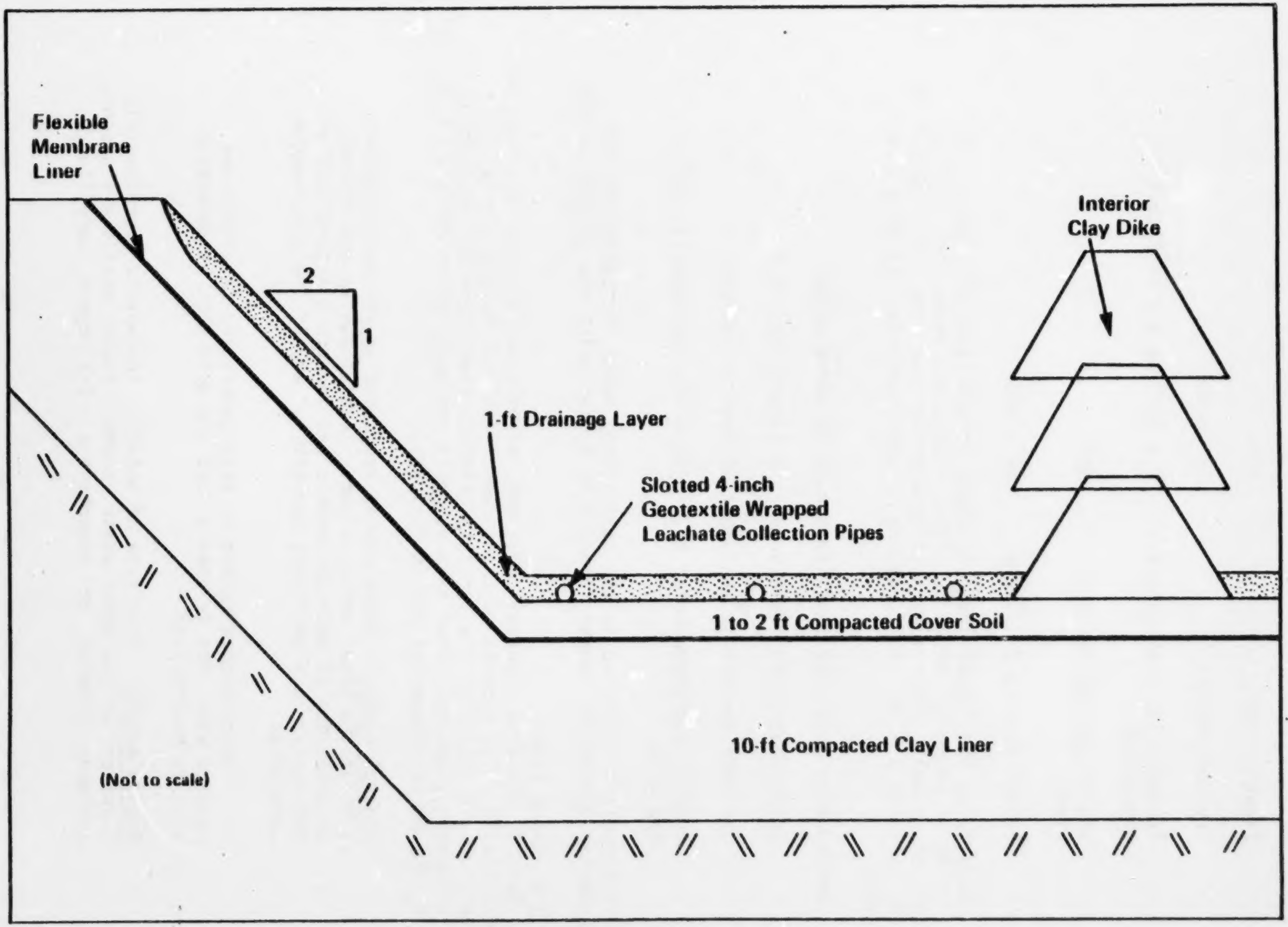

Figure 7-8. Cross-sectional view of site F.

Plastic limit	22%
Dry unit weight	99 lb/ft^3
Permeability (laboratory compacted)	1 x 10^{-8} to 6 x 10^{-8} cm/s
Amount passing No. 200 sieve	99%
Amount passing 2 microns	54 - 63%.

Underlying the lacustrine clay was a 5-foot layer of glacial till consisting of brown to red-brown silts, clays, and sands with varying amounts of gravel. Laboratory testing demonstrated that the permeability of the till was 3 x 10^{-7} cm/s, making it unsuitable for use as a liner material.

Groundwater under the facility occurs in three zones:

- Unconfined water table above the lacustrine clays
- Confined aquifer in the upper 10 feet of bedrock
- Immobilized groundwater held within the impermeable confining beds.

Piezometers installed in the soil-boring holes indicated that the groundwater elevations ranged from 0.5 to 6 feet below the ground surface.

7.2.7.4 Waste Type--

Each of the five disposal cells are, or will be, divided into four or five subcells. The purpose of these subcells is to isolate the various waste groups accepted at the facility, thereby preventing the interactions of incompatible wastes. When five subcells are used, as in cells 1, 2, 3, and 4, the waste categories are as follows:

- General wastes. These wastes represent approximately 44 percent of the total waste volume. General wastes are defined as materials of both an organic and inorganic nature that do not contain a significant quantity of any of the other waste categories.

  The hazardous acidic or acid-generating materials are covered with lime to ensure that any acid that is generated will be neutralized.

- Pseudo metals. This type of material represents approximately 6 percent of the total waste volume. Pseudo metals are arsenic, antimony, bismuth, and phosphorous. Chalcogens, beryllium, and any of their compounds as well as alkaline-sensitive materials are also disposed in this subcell.

This subcell has a pH buffer system that maintains pH levels between 6 and 8.

- Heavy metals. These wastes represent approximately 15 percent of the total waste volume. This group is comprised of all heavy metals and asbestos. This subcell contains the smallest amount of organic materials, which helps to reduce fire hazards caused by the reaction of strong oxidizing agents with organics.
- Highly flammable wastes. This type of material represents approximately 12 percent of the total waste volume. These materials generally exhibit a flash point between 80 and 100 °F. These materials are kept apart from powerful oxidizing agents, materials that are prone to spontaneous heating, or materials that react with air or moisture to evolve heat.
- Toxic materials. These wastes represent approximately 23 percent of the total waste volume. Included in this category are all highly toxic organic compounds, carcinogens, PCB's, and other halogenated wastes. No solvent-type wastes were permitted in this subcell.

If only four subcells are used, the psuedo-metals subcell is eliminated.

7.2.7.5 Liner Description--

The liners at the five secure land disposal cells are very similar. Minor design changes have been made as each cell was constructed. The components of the liners include:

- A minimum of 10 feet of local and borrow soil placed above the in situ glacial till. The soil is compacted so that the permeability is no greater than 1×10^{7} cm/s.
- An FML. Both Hypalon® and HDPE ranging from 30 to 80 mils have been used.
- Cover soil of 12 to 24 inches compacted to a permeability of 1×10^{-7} cm/s or less.
- In some of the cells, an 8-inch layer of slag material placed over the cover soil. This layer is to provide a firm base for equipment movement.
- A leachate collection system located on top of the liner system. This system consists of a 12-inch drainage blanket, slotted 4-inch geotextile wrapped pipes located 100 feet apart (maximum), and standpipes or manholes to enable collection and removal of the leachate.

7.2.7.6 Liner System Installation--

Prior to liner installation, all of the industrial slag material and the organic marsh silt were removed. The lacustrine clays were not excavated because of the high water table. Dewatering was conducted as necessary.

Borrow liner material was transported to the facility. Prior to liner compaction a test patch was required. This patch enabled the construction contractor to determine the compactive effort necessary to achieve the required permeability. Density, moisture content, and permeability measurements were conducted on the test patch to establish the ability of the equipment to compact the liner to the design specifications.

It was determined that the liner should be compacted in 6-inch lifts with a sheepsfoot roller. The finished clay liner thickness was 10 feet. The sidewalls were installed in horizontal lifts and cut back to a final slope of 2:1. The slope of the bottom liner ranged from 1 to 2 percent. The clay liner was protected by the FML. The entire liner was installed before waste placement was initiated.

A system of dikes was used to separate the large cell into the various subcells. The interior dikes were constructed in 4½-foot-high sections. New sections of the interior dikes were constructed by keying into the top of the previous dike (see Figure 7-8).

CQA consisted of soil density and moisture content measurements (nuclear gauge) and laboratory permeability tests on undisturbed Shelby tube samples of the liner.

7.2.7.7 Performance--

The performance of this facility is very difficult to determine at this time. A potential Superfund site located next to this facility has caused extensive groundwater contamination in the area. For this reason, baseline groundwater data have not been obtained. However, it is known that the leachate removed from the collection system of the newly constructed facility is less contaminated than the local groundwater.

7.2.8 Site G

7.2.8.1 Physical Description--

The facility consists of several disposal areas covering 80 acres. A 100-acre section is planned for future development. Some sections of the landfill are active, while others are capped (see Figure 7-9). Each disposal area is lined and/or capped with unrecompacted local soil. A leachate collection system was installed in 1982. Several monitoring wells are located within the area.

7.2.8.2 Startup Date--

This site was first used as a dump in 1955. Sanitary landfilling operations began in 1967. A leachate collection system was added in 1982.

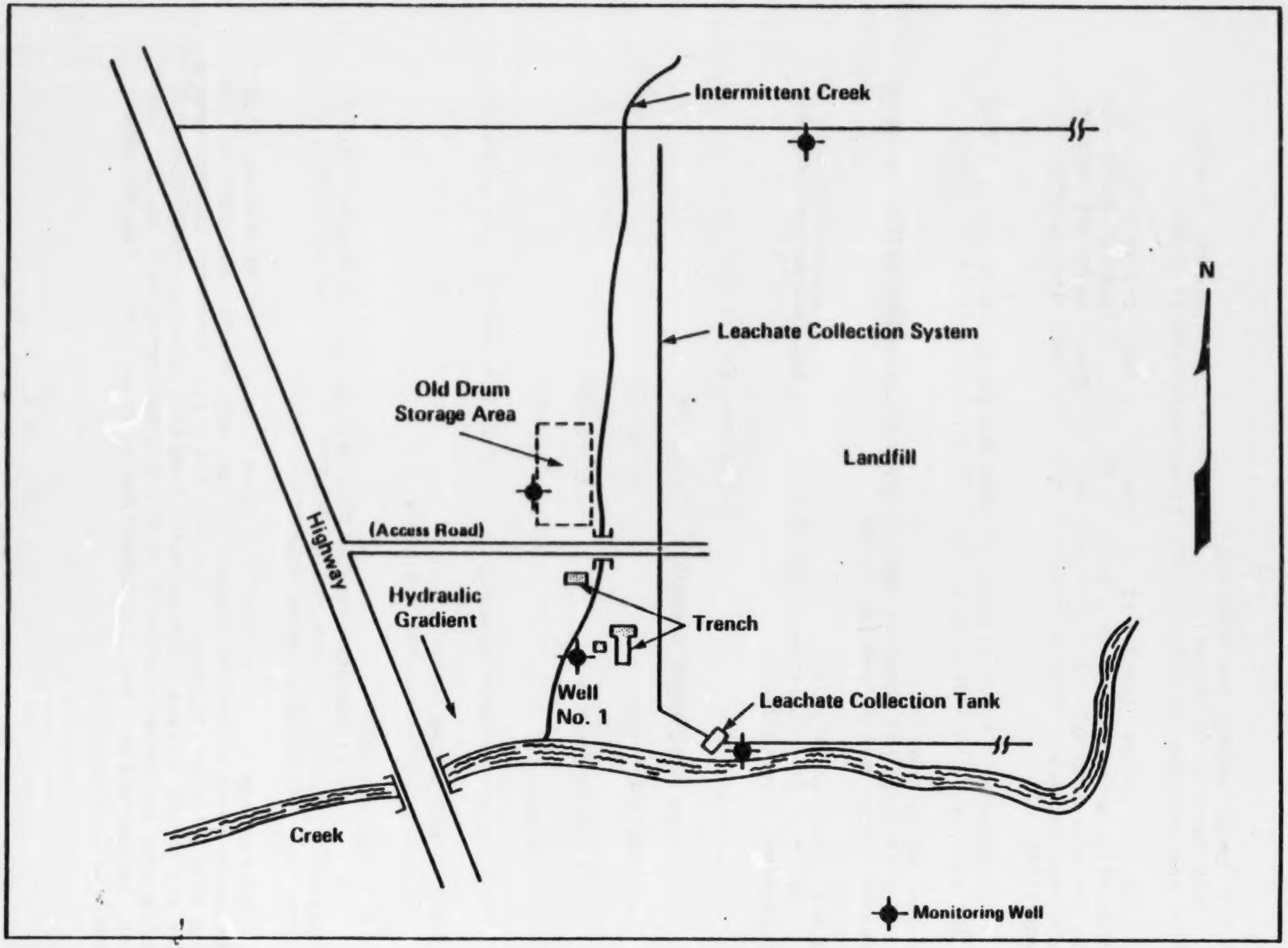

Figure 7-9. Plan view of site G.

7.2.8.3 Local Geology and Hydrology--

This facility is located in the midwestern United States. Average annual precipitation at this facility is approximately 35 inches.

The soil in the vicinity of the landfill is described as silty clay to clayey silt, which extends to a depth of 200 feet. A layer of shale underlies the till material. Discontinuous sand and gravel seams are located throughout the area. When encountered, they have been removed prior to landfilling.

Groundwater is known to occur in sand and gravel seams within 6 feet of the surface. The static water level in the clay soil is not known.

7.2.8.4 Waste Type--

The facility has accepted various types of hazardous wastes including organics, heavy metals, and pesticides.

7.2.8.5 Liner Description--

The local soil (unrecompacted) used to line the site has the following physical properties:

• Permeability	1×10^{-7} to 1×10^{-9} cm/s
• Natural moisture content	9 to 12%
• Plastic limit	20 to 21%
• Liquid limit	11 to 13%
• Plasticity index	8 to 10%
• Cation exchange capacity	1.2 to 2.0 (milleq./100 g sample).

7.2.8.6 Liner System Installation--

This information was not available.

7.2.8.7 Performance--

Analysis of groundwater samples from well No. 1 (see Figure 7-9) has indicated high levels of lead, chloride, COD, total dissolved solids, chloroform, and various chlorinated organics.

This contamination is thought to come from an old drum disposal site previously owned by another company located adjacent to the landfill. The drum disposal site is located on a sand and gravel seam that extends upward near the ground surface. Contaminants from this site can reach this permeable area and travel southward to Site G's monitoring well No. 1. Therefore, indications are that the contamination in well No. 1 has not come from Site G.

7.2.9 Site H

7.2.9.1 Physical Description--

This site has a rectangular 9-acre cell lined with 4 feet of locally obtained clay that was put down in four lifts. A leachate collection system composed of 8-inch perforated polyvinyl chloride (PVC) pipe is on top of the clay liner. Directly below the clay liner are two lysimeters. A third lysimeter is located under the leachate storage basin (see Figure 7-10). The present cell is filled to near capacity, and a new cell is being constructed in such a manner that the bottom clay liners will be tied together to form one large cell with no vertical wall dividing the old and new disposal areas. The presence of lysimeters almost directly below the leachate collection drain pipes presents a unique opportunity to evaluate the performance of the intervening layer of clay.

7.2.9.2 Startup Date--

Operation began in December 1980.

7.2.9.3 Local Geology and Hydrology--

The facility is located in the northern central United States. Average annual precipitation in this area is 30 inches.

The site is underlain by approximately 50 to 100 feet of dense silty sand. This material contains boulders, cobbles, and pockets of sandy material. The permeability of this glacial till ranges from 1×10^{-4} to 9×10^{-7} cm/s. Bedrock is encountered beneath this layer. The water table is at a depth of 50 to 80 feet in the silty sand or bedrock. Groundwater flow is primarily in the southeast direction.

7.2.9.4 Waste Type--

Wastes accepted at this facility consist of approximately 75 percent municipal and 25 percent paper-mill sludge. Small quantities of other industrial wastes are also disposed of at this facility.

7.2.9.5 Liner Description--

The liner system consists of 4 feet of recompacted clay underlain by two collection lysimeters. The clay for the liner was obtained from a nearby borrow area. It consists of a reddish-brown to reddish-gray silty clay. The specifications for the material used were as follows:

•	Liquid limit	>30%
•	Plasticity index	>15%
•	P-200	>50%
•	Permeability	$<1 \times 10^{-7}$ cm/s.

A leachate collection system is present above the clay liner (see Figure 7-11).

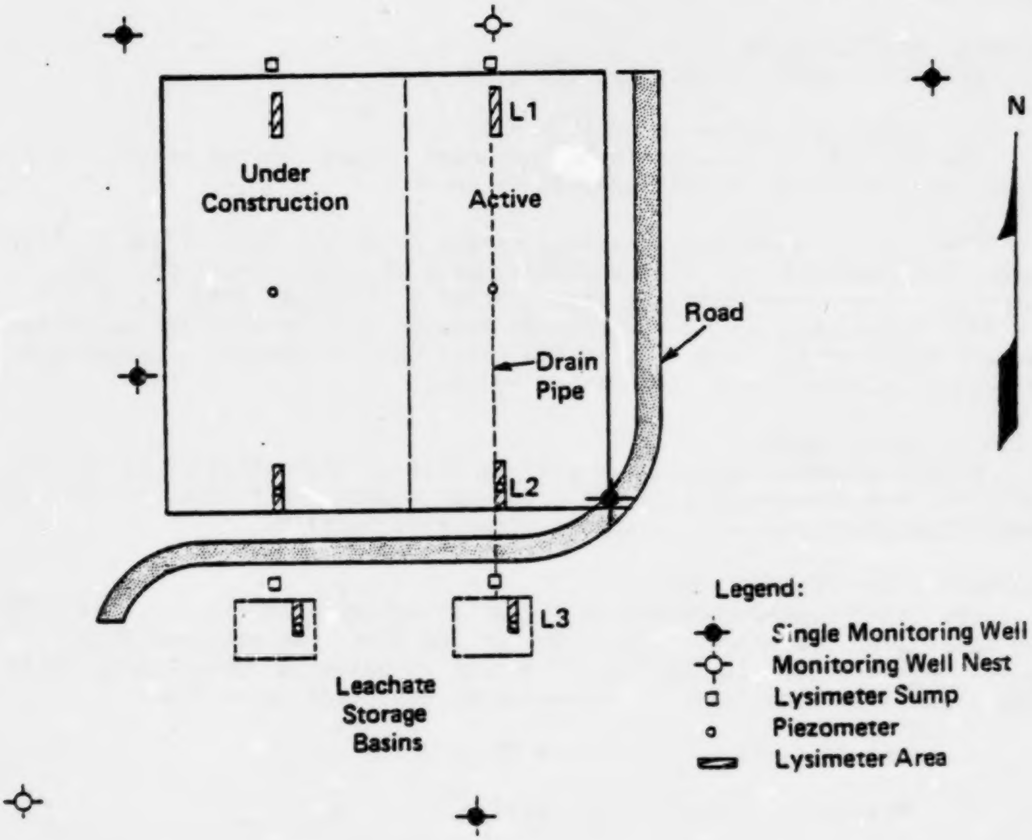

Figure 7-10. Plan view of site H.

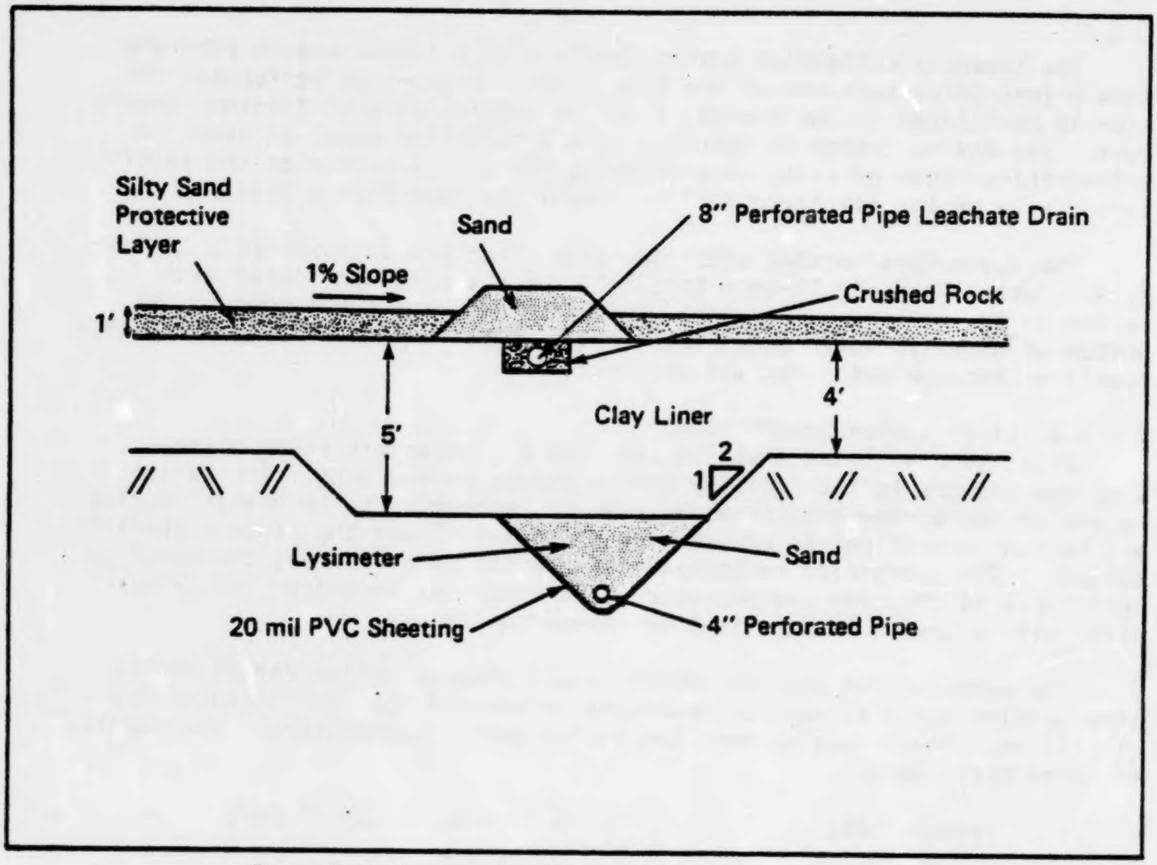

Figure 7-11. Cross-sectional view of site H liner showing details of leachate collection system and lysimeter construction.

The leachate collection system consists of a 1-foot trench excavated into 5-foot-thick sections of the clay liner. Eight-inch perforated PVC pipe is positioned in the trench, which is backfilled with 3/4-inch crushed rock. The entire trench is overlain by a 2-foot-high mound of sand. A 1-foot-thick layer of silty sand covering the entire bottom of the landfill serves as a protective layer for the clay liner (see Figure 7-11).

The lysimeters located under the clay liner are approximately 12 feet by 100 feet. They are lined with 20-mil PVC sheeting and filled with medium to coarse sand. Four-inch perforated PVC pipe positioned in the bottom of each lysimeter (see Figure 7-11) connects them to a manhole, thus enabling leachate detection and collection.

7.2.9.6 Liner System Installation--

Prior to clay liner installation, the fill area was rough graded. Clay was brought to the facility from a nearby borrow area. The variable nature of the borrow area required a soils technician to be present during all removal operations to ensure that the material met the project specifications. The technician rechecked the material by performing the required soil tests as the clay was emplaced. The liner was compacted in 12-inch lifts with either a rubber-tired or sheepsfoot roller.

The construction quality control (CQC) program called for 29 Shelby tube samples taken at various locations in each of the four 1-foot-thick clay lifts. These samples were tested for various parameters. The results of these tests were:

• Permeability	7×10^{-8} cm/s, 5×10^{-10} cm/s
• Liquid limit	39 - 82%
• Plastic limit	20 - 37%
• Plasticity index	16 - 54%
• P-200	>50%
• Density	92 - 102.8% of maximum dry density (nuclear guage) 95 - 110.8% of maximum dry density (sand cone).

7.2.9.7 Performance--

Quarterly sampling of all monitoring points began in December 1980. This sampling included determining leachate volumes and composition, lysimeter liquid volumes and composition, and groundwater composition. The liquid volumes collected from both the lysimeters and the leachate collection system are shown in Table 7-3.

TABLE 7-3. LYSIMETER (L) AND LEACHATE COLLECTION SYSTEM (LCS) LIQUID VOLUMES (GAL) AT SITE H

Date	12/80	3/81	6/81	9/81	12/81	3/82	6/82	9/82	12/82	12/83
L1[a]	38	24	1.9	0.03	Dry	Dry	Dry	Dry	Dry	Dry
L2	66	9.4	0.6	0.03	Dry	Dry	Dry	Dry	Dry	Dry
L3	Dry	1	Dry	1	0.75	Dry	Dry	Dry	Dry	Dry
LCS[b]		NA	NA	89,603	NA	NA	32,995	63,321	89,678	NA

[a] Lysimeter 1 is located below the leachate holding pond.

[b] The numbers indicate the total monthly volume of leachate that was pumped from the collection system at irregular intervals.

NA = Data not available.

As Table 7-3 indicates, the volume of liquid collected in lysimeters has declined steadily over time. The liquid that was initially collected in the lysimeters may be soil moisture released after site construction.

Analysis of leachate and groundwater samples is presented in Table 7-4. To date, there is no apparent difference in water quality between the baseline groundwater samples and the more recently taken samples from upgradient and downgradient monitoring wells.

7.2.10 Site I

7.2.10.1 Physical Description--

This facility consists of three clay-lined surface impoundments as well as a landfill. The ponds that cover approximately 8 acres are lined with two compacted clay liners that are separated by a leak detection system.

7.2.10.2 Startup Date--

Construction of the three ponds at this facility was completed in the fall of 1980. Waste was placed in two of the ponds that fall but was not placed in the third pond until the following spring.

7.2.10.3 Local Geology and Hydrology--

This facility is located in a semiarid section of the western United States. Average annual precipitation at the facility is approximately 15 inches.

Approximately 75 percent of the site area was used for the land application of sewage sludge. This material was disked into the top layer of soil to form a 2-foot layer of highly organic topsoil. Below this layer lies 5 to 7 feet of residual clay soil underlain by 5 to 20 feet of sandstone, which overlays 10 to 20 feet of clay stone bedrock. The residual clay soil has the following properties:

• Liquid limit	36 to 57 (average - 45) %
• Plasticity index	16 to 27 (average - 21) %
• Natural water content	14.6 - 26.1 (average - 23) %
• Dry density	80.9 (1 sample) lb/ft^3
• pH	7.5 - 7.7 (average - 7.6)
• Unified Soil Classification System designation	CH, CL, and CL-CH

The groundwater at the site ranges from 100 to 200 feet below the surface. Drainage across the site is to the east.

TABLE 7-4. MONITORING DATA FOR SITE H

Parameter	Baseline groundwater analysis	Upgradient well	Range for all other wells	Leachate	Range for all lysimeters
Sample date	6/80	9/81	9/81	6/81	6/81
Chloride (mg/L)	1.5 - 5.4	2.5	1 - 3	230	16 - 19
COD (mg/L)	4.0 - 21.0	11	4 - 50	5,315	40 - 57
pH	7.2 - 7.6	7.4	7.2 - 7.4	6.1	7.1 - 7.2
Alkalinity, total (mg/L)	120 - 240	206	150 - 168	960	291 - 445
Conductivity (micro/cm)	300 - 385	405	320 - 350	5,000	625 - 840
Total hardness $CaCO_3$ (mg/L)	122 - 228	222	162 - 182	3,650	344 - 480
Iron, dissolved	0.49 - 2.65	0.06	0.01 - 0.09	161	0.06 - 0.20

7.2.10.4 Waste Type--

The three ponds have received most types of hazardous waste, except pesticides, herbicides, PCB's, dioxin, reactive materials, and any material containing more than 5 percent solvents. Pond No. 1 was the only pond to receive oily wastes.

Wastes were placed in the ponds through a trough leading to the base of each pond.

7.2.10.5 Liner Description--

The liner at this facility consists of two 5-foot layers of recompacted local clay soil that are separated by a 15-inch leak detection layer. The three ponds are separated by dikes that are 12 feet wide at the top (see Figure 7-12). The interior slopes of the dikes (sidewalls) are 3:1. A cross-sectional diagram of the liner system is illustrated in Figure 7-13.

Properties of the local clay soil that was used as the liner material are listed in Section 7.2.10.3. Additional laboratory testing of this material indicated that it had a permeability ranging from 2×10^{-9} to 4.8×10^{-8} cm/s. The value for the permeability of the claystone material ranged from 3×10^{-9} to 1×10^{-8} cm/s.

7.2.10.6 Liner System Installation--

The local soil was excavated with dozers and scrapers. Excavated clayey materials were placed on a stockpile. Here the material was spread in 8-inch lifts and then watered to achieve a moisture content between optimum and 3 percent above optimum. After the excavation was complete, the pond bottoms were graded according to design specifications. This included the additional excavation of a 10-foot-wide trench area beneath the leak detection system collection pipes.

The stockpiled liner material was disked and then placed in the pond excavations in horizontal lifts (which, before compaction, were 8 inches thick). The lifts were compacted to 95 percent standard Proctor density with a sheepsfoot roller.

A backhoe was used to excavate a shallow trench into the bottom liner for the leak detection drain and sump. Perforated pipe was placed in the trench and then covered with gravel. The entire bottom liner was then covered with a sand drainage layer.

The final 5-foot (minimum) clay liner was constructed through the same procedure as previously discussed. A 2-foot protective layer of sandy soil was placed on top of the finished clay liner. Waste unloading troughs were protected at the bottom with rip-rap.

7.2.10.7 Performance--

The construction of the liners of these ponds was completed in the fall of 1980. Ponds No. 1 and 3 were filled with waste shortly thereafter. Pond No. 2, however, was left unfilled and uncovered until the following

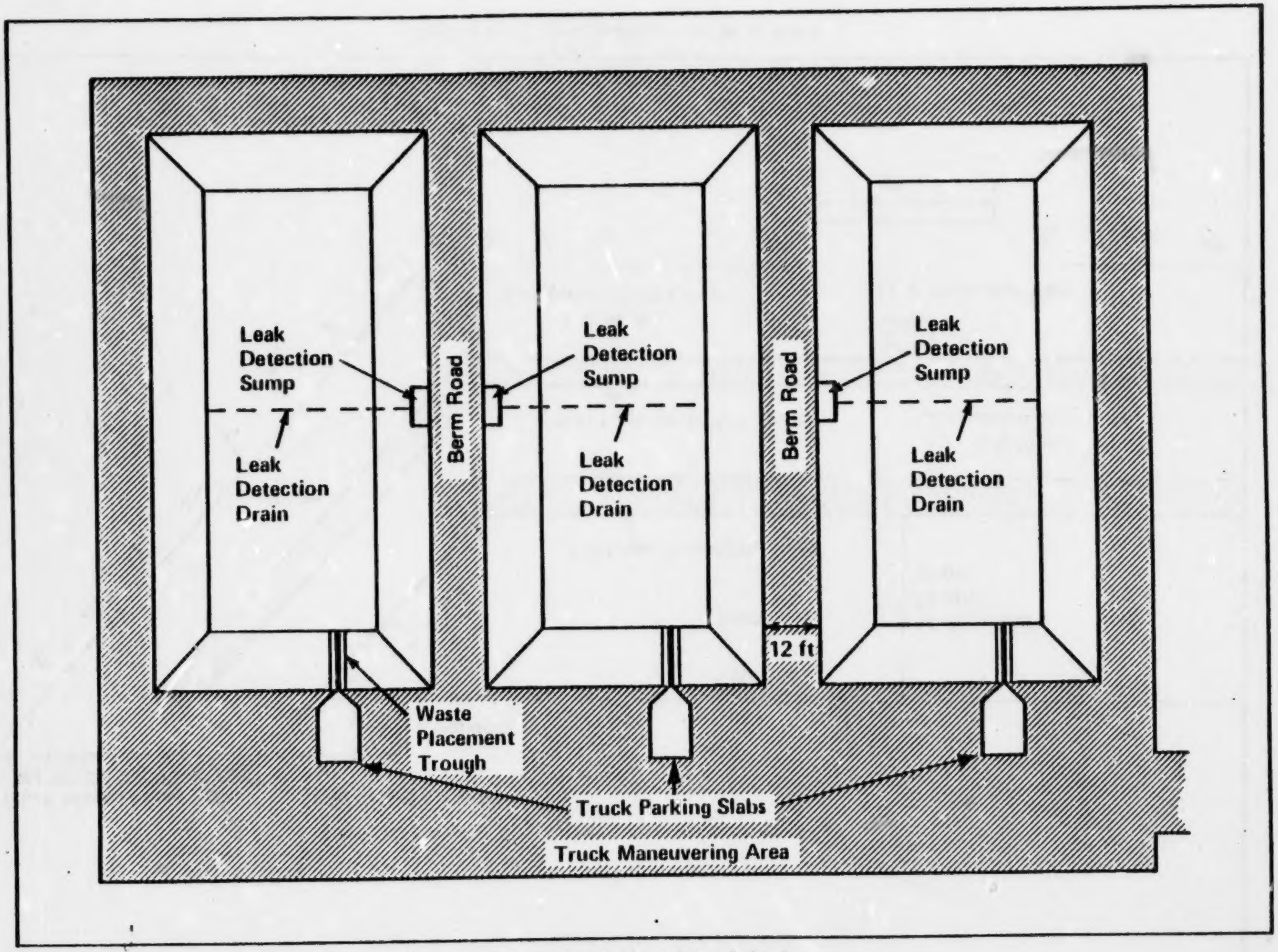

Figure 7-12. Plan view of site I.

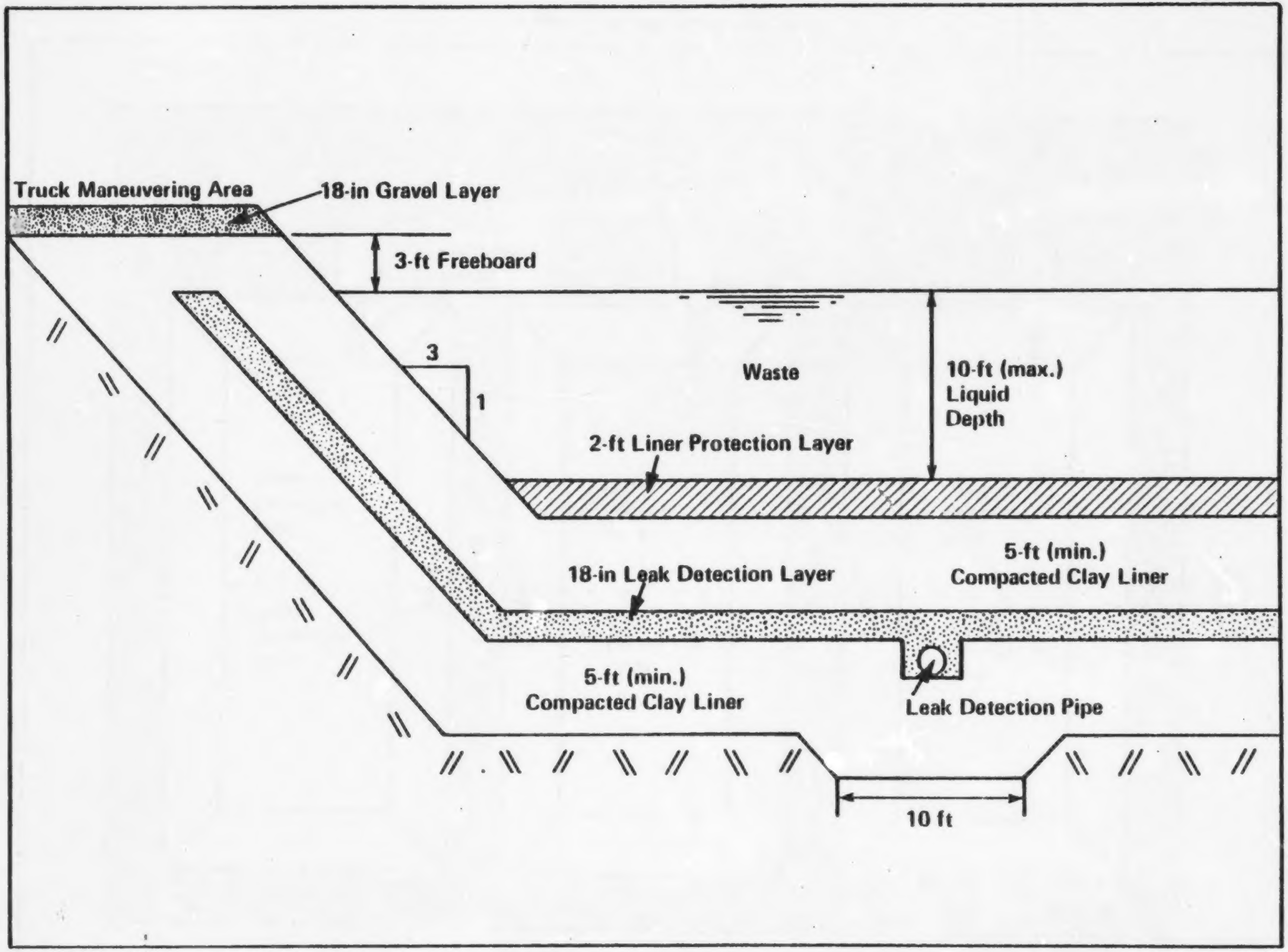

Figure 7-13. Cross section of liner at site I.

spring. Approximately 3 months later, liquid started accumulating in the leak detection system. This liquid had elevated levels of chloride, ammonium, and total dissolved solids as well as increased conductivity. Liquid was not detected in the sumps of ponds No. 1 and 3.

The most probable cause of the failure at pond No. 2 was waste migration through desiccation cracks in the upper liner. Cracks ranging from 2 to 10 mm wide were observed on the upper portion of the sidewalls. In most cases, these cracks could not be traced for any more than a few feet along the surface. An exception to this was in the corners of the liner, where the cracks appeared to be up to 20 feet long.

7.2.11 Site J

7.2.11.1 Physical Description--

This facility consists of a drum disposal pit, a solvent recovery facility, and seven ponds, one of which is used for landfilling. The ponds range from approximately 1/2 to 4 acres. All of these areas are lined with two layers of compacted clay separated by a leak detection system. A 10-foot-high retention dike located northeast of the disposal areas prevents any overflowing liquid, such as may occur during a 100-year storm, from leaving the facility. Several groundwater monitoring wells are installed around the facility (see Figure 7-14).

7.2.11.2 Startup Date--

Ponds No. 1 through 4 and the drum pit went into service in June of 1979. The remaining ponds (No. 5 and 6) started operation in May of 1981. Pond No. 7 was used as a landfill starting in May of 1981.

7.2.11.3 Local Geology and Hydrology--

This facility is located in the southern central United States. The average annual precipitation at the facility is approximately 26 inches.

The top layer of soil at this site consists of red silty clay. Thin sandstone and gypsum lenses are present at depths ranging from 8 to 18 feet below the ground surface. A layer of gray siltstone encountered at depths ranging from 14 to 27 feet underlies the silty clay. The physical properties of the silty clay are as follows:

• Soil type	Very fine silts or clays
• Liquid limit	31.0 - 46.0%
• Plastic limit	22.3 - 29.5%
• Plasticity index	7.4 - 20.1%
• Optimum moisture content	18.0 - 23.7%

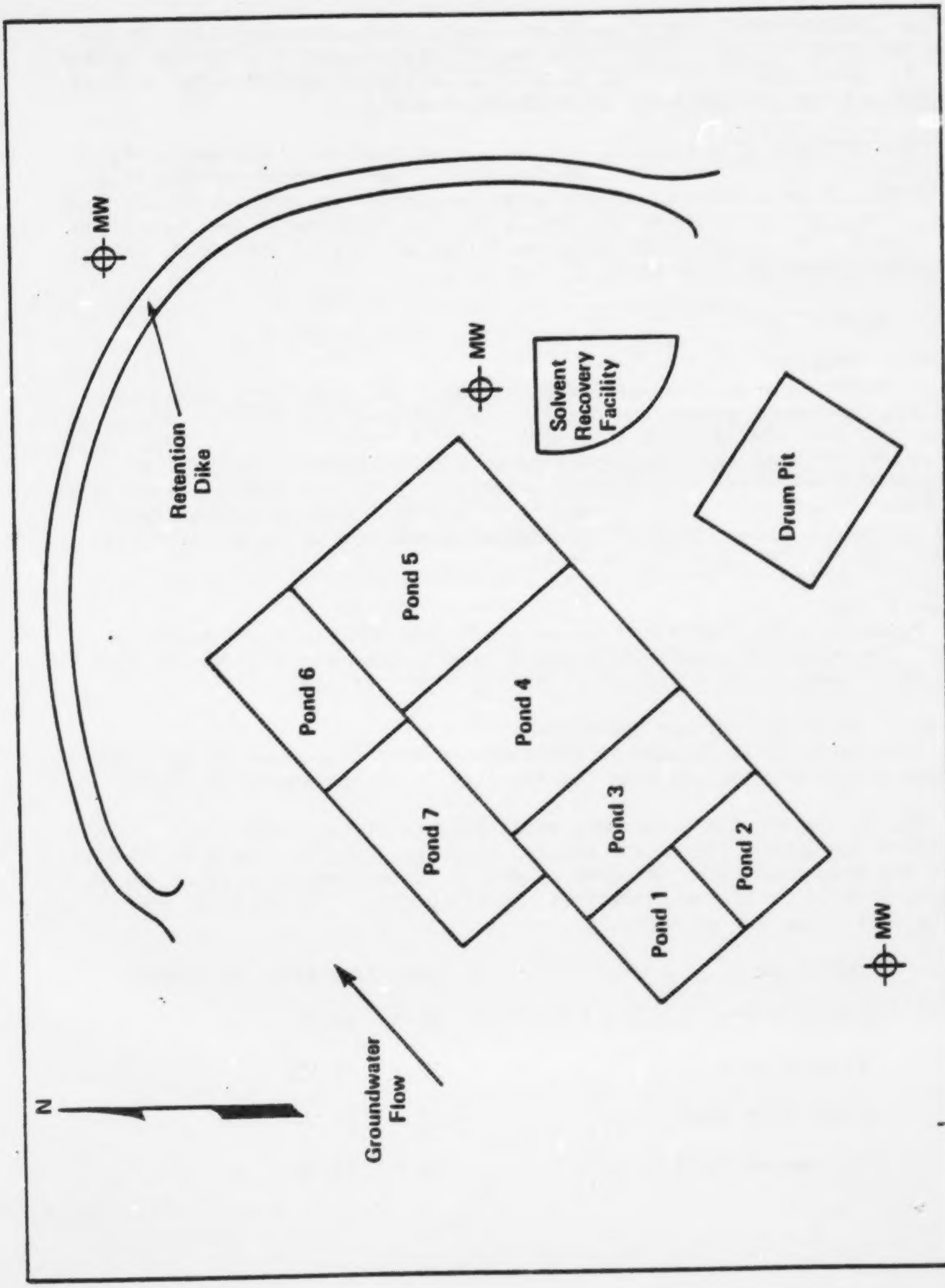

Figure 7-14. Plan view of site J.

• Maximum dry density	99.3 - 110.4 lb/ft^3
• Laboratory permeability	2.0×10^{-9} - 2.9×10^{-8} cm/s.

Six borings were dug in an attempt to determine the location of the water table. At the time of exploration, water was encountered in two of the borings at depths of 7 and approximately 27 feet below the ground surface. Ten days later, water was standing in all six borings at depths ranging from 5 feet 8 inches to 23 feet below the ground surface. Surface drainage from the site is in two directions--north and east.

7.2.11.4 Waste Type--

This facility receives most types of hazardous liquids and sludges.

7.2.11.5 Liner Description--

The bottom liners at this facility consist of two recompacted clay layers separated by a leak detection system. The lower clay liner is 1 foot thick and the upper clay liner is a minimum of 5 feet thick. The sidewalls are constructed partially underground and partially above ground. The aboveground sections are built upon dikes. The leak detection system is a 6-inch drainage blanket that slopes to a trench containing a 2-inch slotted PVC collection pipe (See Figure 7-15). Quality assurance test results of the completed liners are as follows:

• Dry density	94.2 - 115.8 lb/ft^3
• Moisture content	15.8 - 31.0%
• Permeability	5.3×10^{-9} - 2.8×10^{-8} cm/s
• Liquid limit	35.6 - 63.0%
• Plastic limit	16.1 - 31.6%
• Plasticity index	6.7 - 36.6%.

7.2.11.6 Liner System Installation--

No specific information on the liner construction or the equipment used was available. However, it is known that construction of the second phase (ponds No. 5, 6, and 7) took place during the winter. Problems due to frozen and unworkable liner materials caused many construction slowdowns.

Quality control (QC) and quality assurance (QA) inspectors were present at the facility during construction operations. They conducted tests, made observations, and prepared a project diary and final documentation report.

7.2.11.7 Performance--

Liquid, presumed to be construction water, was being collected and removed from all of the detection systems by early 1982. This liquid was periodically analyzed and found to be "clean" until February 1982. At this

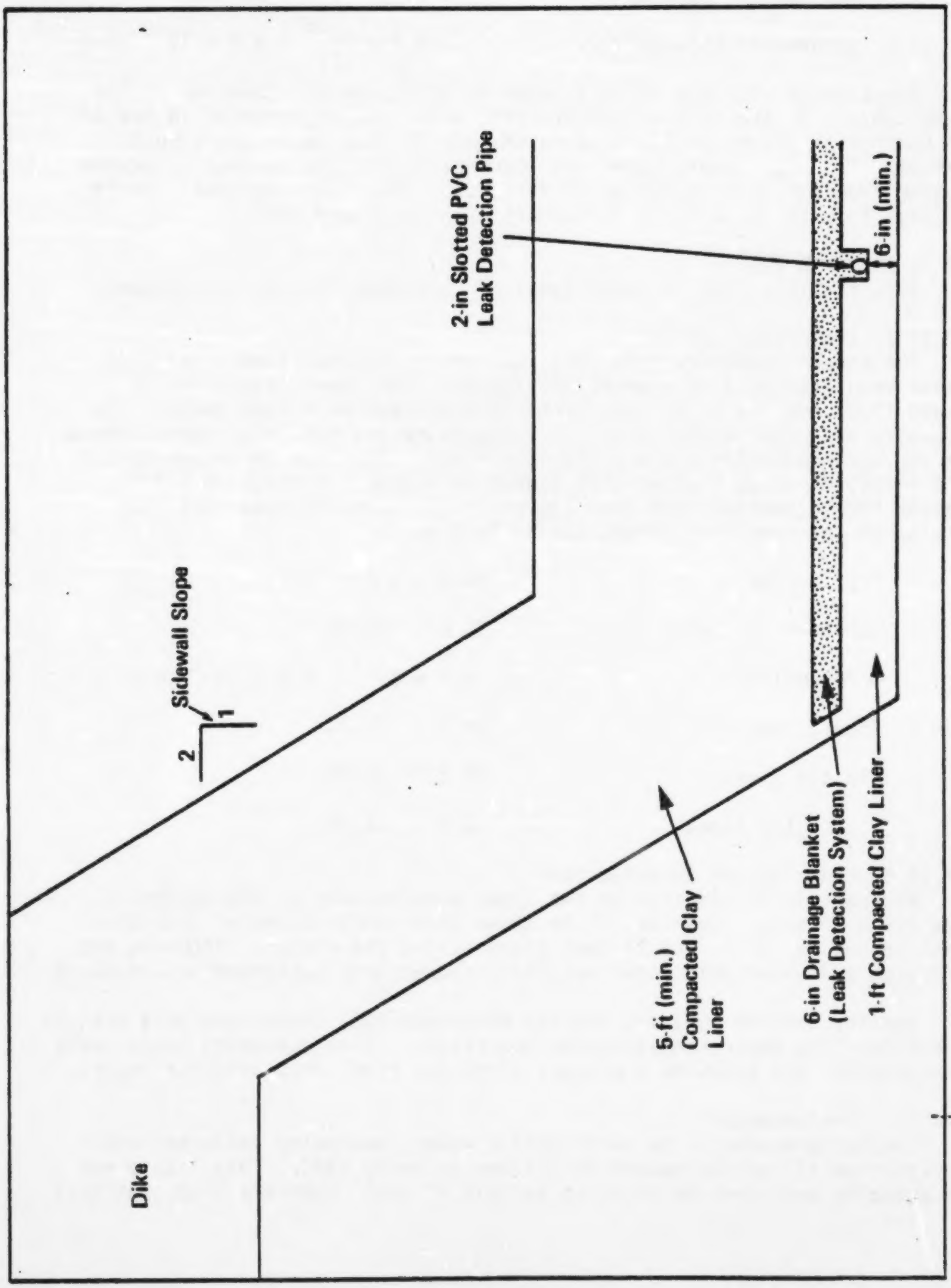

Figure 7-15. Cross-sectional view of site J liner.

time, samples removed from the pond No. 5 detection system contained a yellow-brown oily liquid with an organic-solvent odor. A small water phase was present in the sample. Analysis of this sample showed that it contained over 11.2 percent perchloroethylene (perc) plus small amounts of other chlorinated and nonchlorinated organics. The facility was allegedly not accepting wastes with more than a trace of "perc."

An investigation of the problem revealed that aqueous wastes coming from one of the disposal facility's customers contained approximately 5 percent degreasing fluid waste, which contained perchloroethylene. This waste was to have been stored in a separate tank at the originator's plant. Evidently, it was not made clear to the workers where to put the "perc" waste, and it was placed in an aqueous waste storage tank. Being insoluble and heavier than water, it sank to the bottom. When a tank truck from the disposal facility would collect a load of aqueous waste, it would also receive a few hundred gallons of the "perc" waste. The waste sampling method used by the disposal facility on all loads of incoming waste was not able to sample the bottom few inches of each load. In addition, the parking area where all truckloads of waste were sampled was slightly sloped; this sloping caused the relatively small amount of "perc" waste in the bottom of the tanker to flow to the back of the truck, where it could not be reached by the sampling device.

Over approximately 9 months, the aqueous waste containing the "perc" was placed in one of the clay-lined ponds and went unnoticed until the liner failure occurred. The apparent incompatibility is consistent with current research on the effects of chlorinated solvents on clays.

Presently, the leak in the pond No. 5 liner has slowed down considerably. Perchloroethylene concentrations in the detection system liquid are less than 100 ppm. It is estimated that several thousand gallons of the "perc" waste are adsorbed into the clay liner and the sludges in the pond's bottom and will present minor problems for many years.

7.2.12 Site K

7.2.12.1 Physical Description--

This facility consists of six double-lined ponds ranging in size from 1.2 to 2 acres. The liners are composed of a 3-foot recompacted clay layer, below which lies a leak detection system and a 1-foot layer of recompacted clay. A typical cross-section of the pond and liner system is illustrated in Figure 7-16. The design depth of all six ponds is 10 feet.

7.2.12.2 Startup Date--

The construction of the first pond at this facility was completed in December 1981. Five additional ponds have been constructed since then.

7.2.12.3 Local Geology and Hydrology--

This facility is located in the western United States. Average annual precipitation at this facility is approximately 6 inches, while the average annual evaporation is 63 inches.

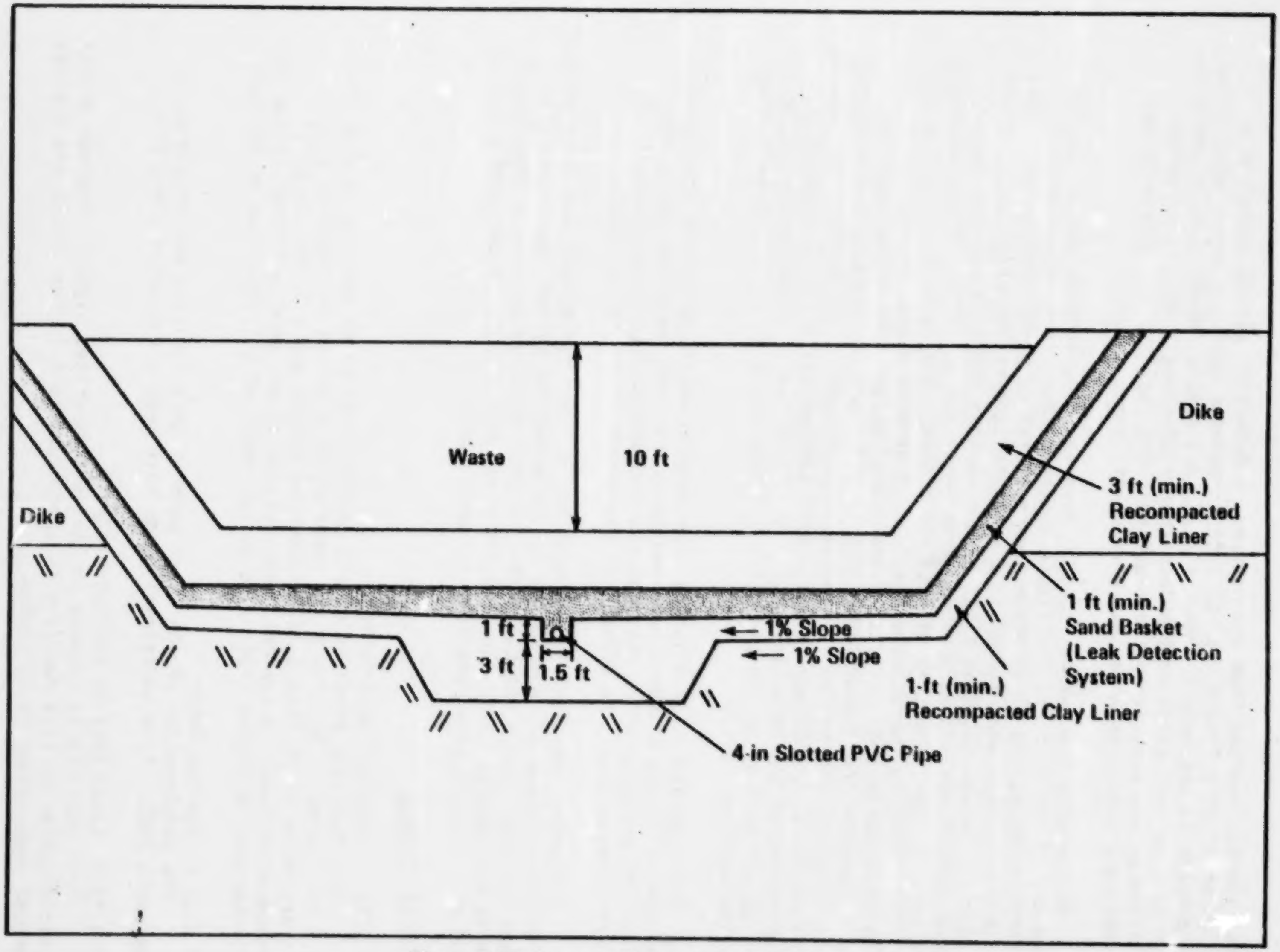
Dike
Waste
10 ft
Dike
3 ft (min.) Recompacted Clay Liner
1 ft (min.) Sand Basket (Leak Detection System)
1% Slope
1% Slope
1-ft (min.) Recompacted Clay Liner
1 ft
3 ft
1.5 ft
4-in Slotted PVC Pipe

Figure 7-16. Cross-sectional view of site K liner.

A total of 10 trenches and 12 borings ranging from 8 to 100 feet in depth were used to investigate the geology at this facility. This investigation, combined with prior knowledge, revealed three distinct layers of silty claystone, siltstone to sandstone, and silty claystone. The uppermost layer caps a flat ridge at the facility. Here the silty claystone is up to 80 feet thick. The next layer ranges up to 120 feet thick. The lower layer is in excess of 80 feet thick across the entire site.

The uppermost claystone layer was characterized most extensively. It has the following properties:

•	Liquid limit	56 - 84%
•	Plasticity index	36 - 55%
•	P200	83 - 99%
•	Laboratory permeability (remolded)	5.0×10^{-7} - 2.8×10^{-8} cm/s
•	Laboratory permeability (undisturbed)	3.7×10^{-7} -6.1×10^{-8} cm/s

Investigations also revealed the presence of occasional gypsum veins throughout the claystone material. These veins ranged in thickness from less than 1/10 inch to about 3 inches and had an average permeability of 2.4×10^{-3} cm/s.

A review of published data indicated that several seismic faults exist in the region of the disposal facility. However, within 4 miles of the site no seismic faults show evidence of recent creep. The two seismic faults within 1/2 mile of the facility are both inactive.

State investigations indicate that groundwater is scarce in the vicinity of the site. This fact was confirmed by drilling 12 borings on the site property. Of the 12, only 1 encountered water at a depth of 32 feet perched on claystone bedrock.

The scarcity of groundwater is due to low rainfall, high evaporation, and the fine-grained nature of the sediments.

7.2.12.4 Waste Type--

The facility accepts liquid-scrubber wastes that are generated while stack gases are cleared from oil-refining facilities. The wastes are highly saline with pH values in the wide range of 3.5 to 9.0. The waste is temporarily stored in the evaporation ponds. When it has been reduced to a semisolid, it is removed and disposed of onsite.

7.2.12.5 Liner Description--

The pond liners were constructed of the local excavated claystone material. The liner system consists of a 1-foot recompacted basal liner. Sump and collection trenches are excavated into this bottom liner. A

1-foot sand layer covers the lower clay liner. The sand layer is overlain with 3 feet of clay.

Four undisturbed samples were taken from one of the constructed liners and tested for their Atterberg limits, percent passing the No. 200 sieve, moisture content, and dry density. The results of these tests are presented below.

	1	2	3	4
• Atterberg limits				
-Liquid limit (%)	70	87	84	93
-Plasticity index (%)	49	67	64	69
• Amount passing No. 200 sieve (%)	86.9	88.1	88.3	87.2
• Moisture content (%)	24.9	21.5	20.8	23.0
• Dry density (lb/ft^3)	94.6	103.0	104.4	99.8

Permeability tests were conducted in which remolded liner samples were subjected first to water and then to typical wastes. The permeability of the liner material to both water and the scrubber waste ranged from 2×10^{-7} to 1×10^{-8} cm/s. The design specifications called for the liner permeability to be no greater than 1×10^{-6} cm/s.

7.2.12.6 Liner System Installation--

The leak detection system was constructed by first excavating and subsequently recompacting the clay soil to provide a 3-foot foundation beneath the collection drains and sump. A 1-foot clay liner was then placed over the base and side slopes of the excavated area. The sump and drain-pipe trenches were then excavated over the 3-foot foundations. After the installation of the drainage system, a 1-foot layer of sand was placed over the entire clay liner. Finally, a 3-foot recompacted clay liner was placed on top of the sand layer. A segmented steel-wheel compactor was used for liner compaction.

Construction activities were inspected by the design firm. Included in these inspections were numerous density, moisture content, and permeability tests. At the completion of construction, the design firm certified that the facility was constructed according to design specifications and would therefore perform as designed.

7.2.12.7 Performance--

Liquid volumes collected in the leak detection system were used to determine the installed liner permeability. The permeabilities for five of the six ponds were calculated based on Darcy's Law, the collected leachate volumes, and the impounded liquid depth and pond area. The value for pond No. 4 was not available. These values are as follows:

Pond	Average Permeability (cm/s)
1	2.95×10^{-7}
2	1.8×10^{-7}
3	4.1×10^{-8}
4	Data not available
5	1.4×10^{-7}
6	1.4×10^{-7}

The permeability of the installed liner system is less than the specified value of 1×10^{-6} cm/s.

This facility is located approximately 100 miles south of the epicenter of a recent major earthquake. After the quake, the excavation, slopes, and berms were examined carefully, but no indications of seismic damage were noted. Careful monitoring of the ponds since the quake has not revealed any unusual changes in fluid levels or leachate collection volumes.

7.2.13 Site L

7.2.13.1 Physical Description--

The landfill consists of a flat double-clay liner on top of which a dike has been placed to contain the wastes. The liner extends beyond the dike to form the liner for a 12-foot-wide drainage ditch that encircles the site (see Figure 7-17).

The landfill is divided into two separate sections, one for each of two types of waste. The largest area is designed to contain dewatered sludge pumped from settling tanks. It has a 5-foot-high perimeter dike and a drainage blanket on top of the liner composed of 1 foot of graded clean sand with drain pipes installed to collect and remove leachate. Sludge is transferred to this portion of the landfill through a pipeline from offsite settling tanks.

The smaller section of the landfill is designed to hold chemical wastes from the plant manufacturing units and QC laboratory. It has a gravel bottom over its liner. The working area was developed in sections across the width of the landfill with the waste layer maintained at approximately 3 feet in depth. Each lift of waste was covered with river sediment material previously deposited on the site.

7.2.13.2 Startup Date--

The site became active in 1978.

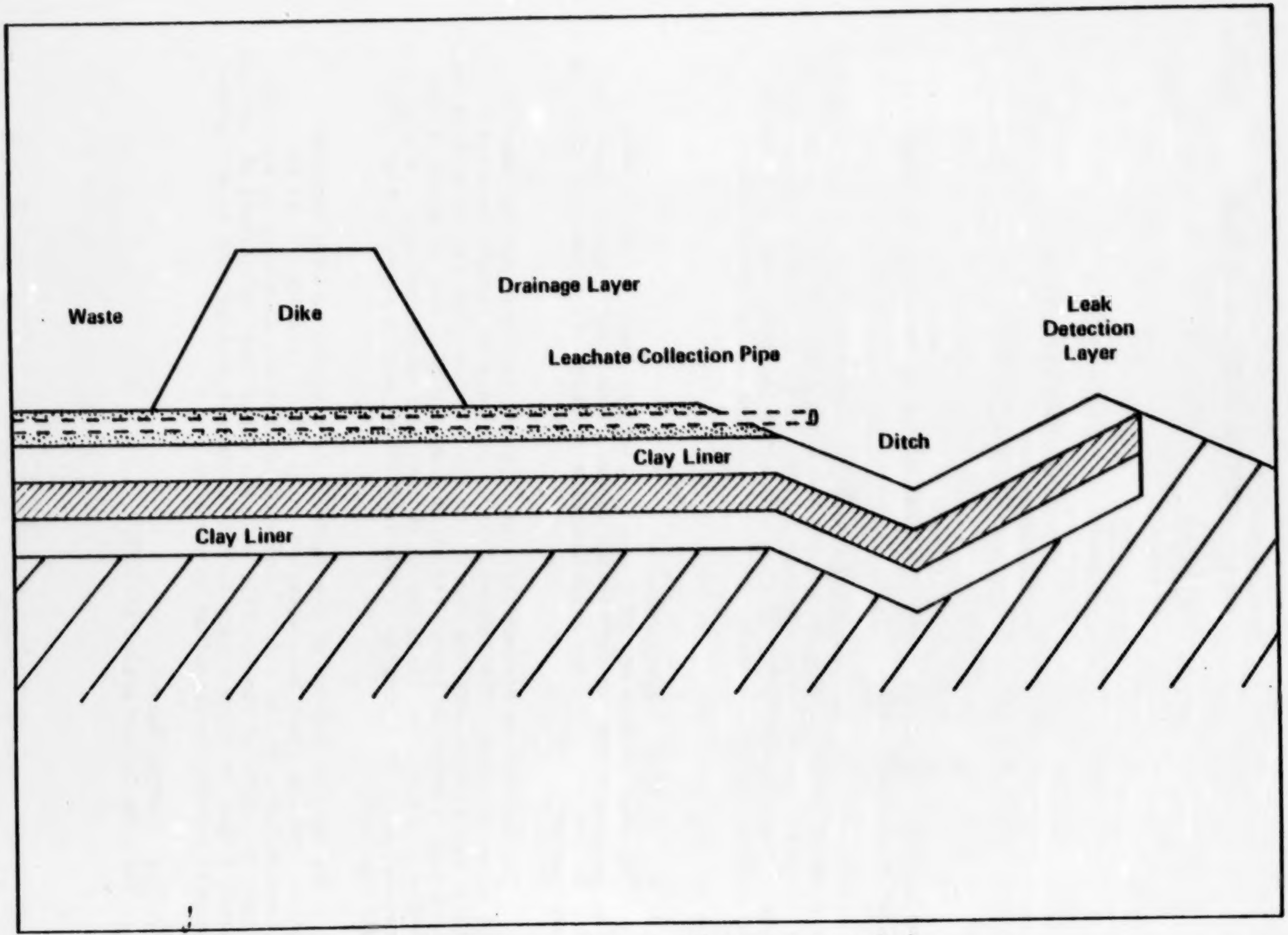

Figure 7-17. Cross-section of containment system at site L.

7.2.13.3 Local Geology and Hydrology--

The facility is located in the northeastern United States. Average annual precipitation at this facility is approximately 40 inches.

The facility is located in an area where there is approximately 3 feet of topsoil over a subsoil varying from silt to sand to fine gravel and sandy clay. Groundwater is located in the subsoil layer at depths ranging from 4.5 to 14.5 feet below the ground surface.

7.2.13.4 Waste Type--

The landfilled waste material can be defined as either sewage treatment plant sludge or chemicals. The sewage treatment plant sludge consists of dewatered (12 to 15 percent solids) sludge removed from settling tanks and transferred to the landfill. The chemical waste consists of inert material from lime slaker operations (inert rocks and insoluble calcium and magnesium salts), hard-pitch residue that is only slightly water soluble and that crystallizes at 180 °C, filter aid (dicalite) wetted with phosphate esters, filter paper wetted with phthalate esters, and residue from the manufacture of tetrachlorophthalic anhydride.

7.2.13.5 Liner Description--

The liner system at this facility consists of the six layers listed below from the top down:

- 8- to 12-in. drainage layer (sand or gravel)
- 18- to 25-in. compacted clay
- 12-in. sand leak detection layer
- 12-in. compacted clay
- 6- to 18-in. compacted soil
- Bidim type C34 (synthetic soil stabilization geotextile).

The physical properties of the clay used for the liner are as follows:

- Soil classification - grey silty clay

Particle size distribution:	Gravel - 1% Sand - 29% Clay and colloids - 70%
Liquid limit	45%
Plastic limit	17%
Plasticity index	18
Liquidity index	0.1

Specific gravity	2.67
Moisture content	28.5%
Dry unit weight	95.0 $\mathrm{lb/ft^3}$
Maximum dry density	101.5
Optimum moisture content	19.5%
Permeability	6.6×10^{-8} to 3.8×10^{-9} cm/s.

7.2.13.6 Liner System Installation--

No information was available on installation procedures.

7.2.13.7 Performance--

Within the first year of operation, problems developed in the upper clay liner. Evidence of this was the appearance of contaminated water in the leachate monitoring layer manhole. Slight contamination in local groundwater monitoring wells was also discovered. A study conducted to determine the source of the groundwater contamination revealed that the source of contamination was not the double-lined pond.

7.2.14 Site M

7.2.14.1 Physical Description--

This 12-acre site is designed to consist of three cells of approximately equal size. At the time of this writing, the first cell was still in use and construction had not yet started on either of the remaining two cells.

The facility is lined with 1 foot of recompacted clay. A leachate collection system is above the clay liner and a leak detection system lies 2 feet below the clay liner. In addition to the leachate collection and subsurface monitoring (leak detection) systems, three groundwater monitoring wells are situated around the facility as well as drainage ditches to prevent surface runoff from entering the landfill (see Figures 7-18 and 7-19).

7.2.14.2 Startup Date--

Construction was initiated in 1977. The first wastes were accepted in 1978.

7.2.14.3 Local Geology and Hydrology--

This facility is located in the northern central portion of the United States. Average annual precipitation in the vicinity of this facility is approximately 26 inches.

A total of five borings were used to investigate the subsurface conditions at the proposed landfill site. The borings indicated a uniform geologic profile over the entire facility. The top 1 to 2 feet of soil consists of a silty sand underlain by approximately 25 feet of fine- to medium-grained sandy alluvium.

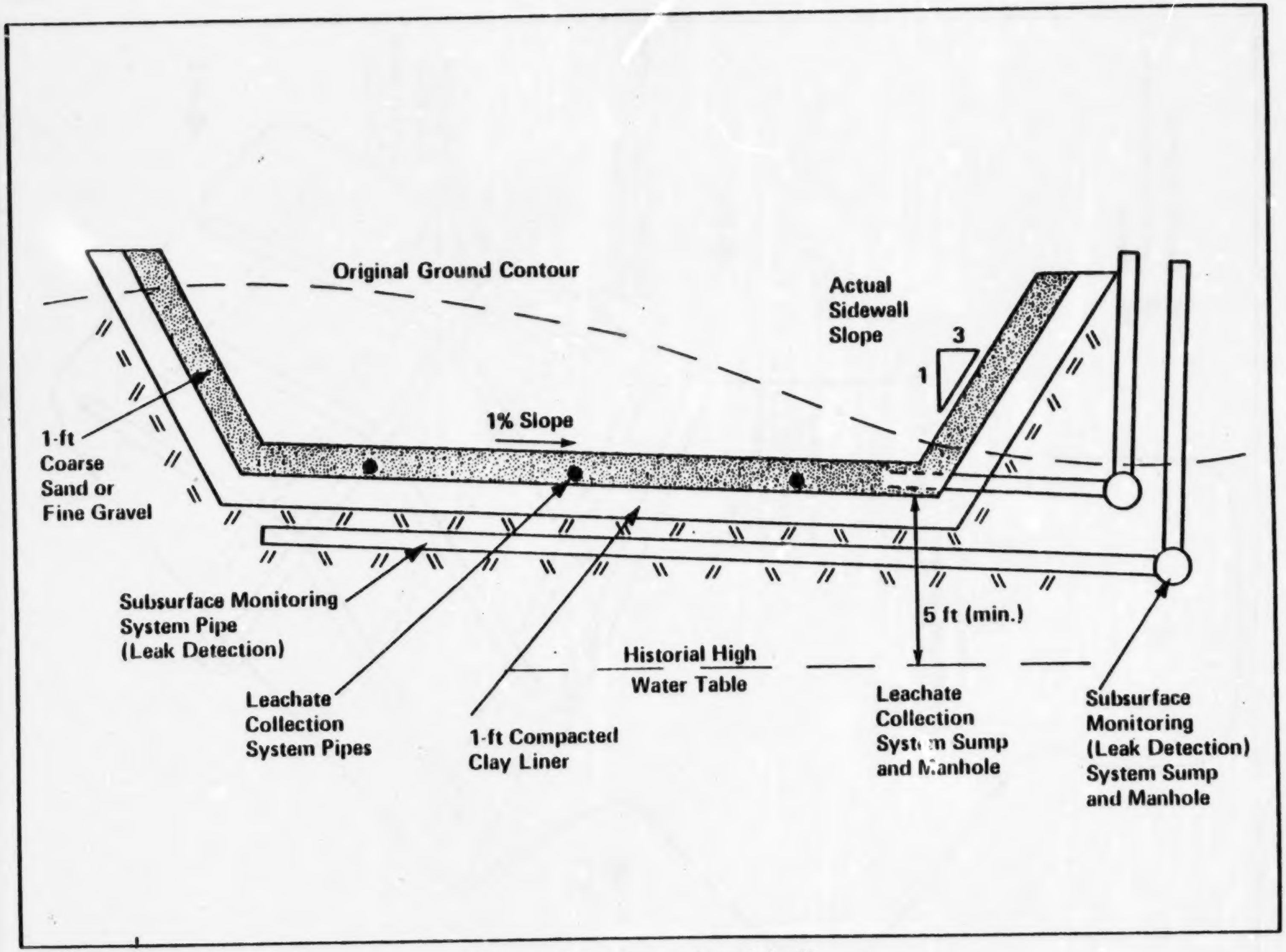
Original Ground Contour
Actual Sidewall Slope
3
1
1% Slope
1-ft Coarse Sand or Fine Gravel
Subsurface Monitoring System Pipe (Leak Detection)
Leachate Collection System Pipes
1-ft Compacted Clay Liner
Historial High Water Table
5 ft (min.)
Leachate Collection System Sump and Manhole
Subsurface Monitoring (Leak Detection) System Sump and Manhole

Figure 7-18. Cross-section of site M.

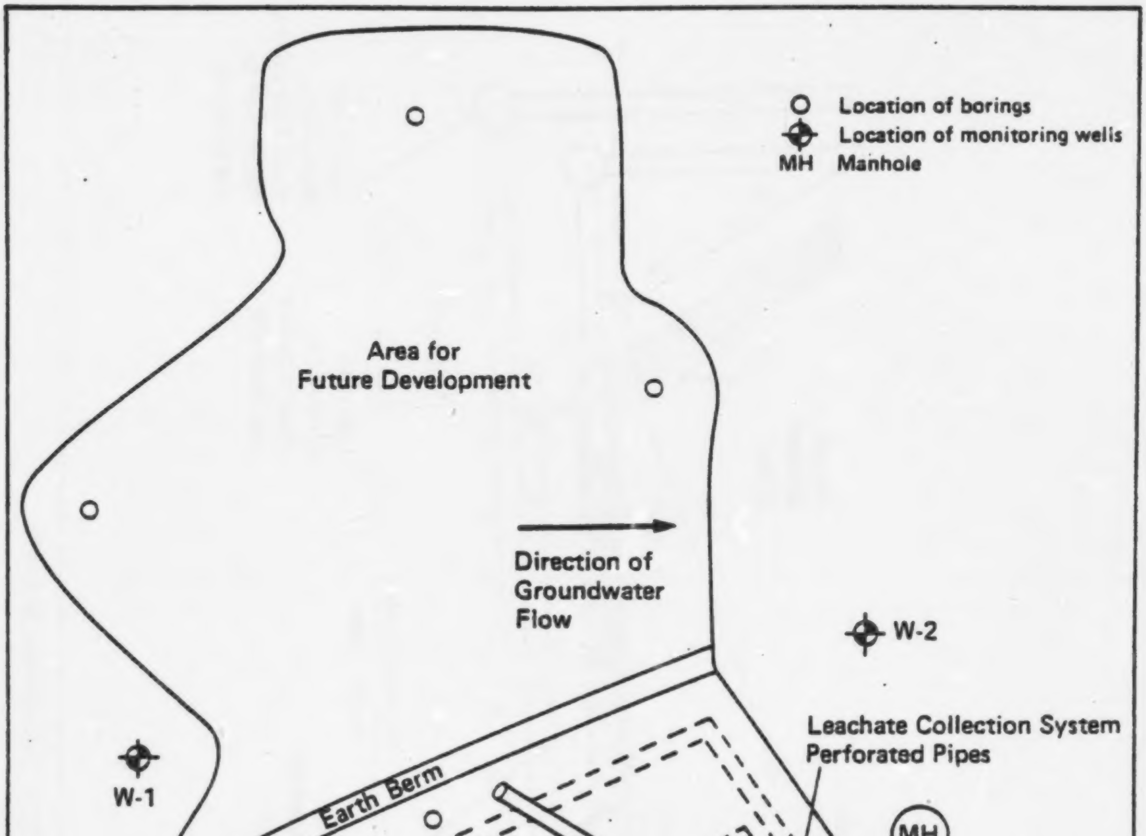
○ Location of borings
Location of monitoring wells
MH Manhole
Area for Future Development
Direction of Groundwater Flow
W-1
W-2
W-3
Earth Berm
Leachate Collection System Perforated Pipes
MH
MH
MH
Subsurface Monitoring System (Leak Detection)

Figure 7-19. Plan view of site M leachate collection and leak detection systems.

Groundwater was encountered in all five borings at depths ranging from approximately 10.5 to 15 feet below the original ground surface. Groundwater flow was toward a marshy area located east of the proposed facility. Other information indicates that in a second deep aquifer groundwater flow is in a southeasterly direction.

7.2.14.4 Waste Type and Placement--
The landfill is used for the disposal of inorganic lime sludge. Samples of the sludge have been analyzed for heavy metals content and percent solids. The results of these analysis are presented in Table 7-5.

Due to the high moisture content of the waste material, a special procedure for waste placement was developed. This procedure involved mixing a thin layer of the sandy soil obtained from the site excavation with a thin layer of the waste in order to reduce the overall moisture content and to give the material structural stability. The range of soil to waste ratios varied from 0.5:1 to 1.5:1. Exact mixing proportions were determined in the field and judged sufficient when the landfill equipment was able to drive over the mixture. Due to the great amount of soil mixed with the waste and the waste's inorganic nature, no daily cover was used at this facility. The proposed final cover will consist of a 6-inch compacted clay cap, 2 feet of topsoil, and a vegetative cover. No gas venting or collection will be necessary.

7.2.14.5 Clay Liner Description--
The clay for the liner was obtained from a nearby borrow area. The specifications for the clay are as follows:

•	Liquid limit	50 - 70%
•	Plasticity index	>28%
•	P200	>75% (by weight)
•	Permeability (recompacted)	$\leq 1 \times 10^{-7}$ cm/s

No documentation was available on the as-built liner material characteristics.

7.2.14.6 Liner System Installation--

7.2.14.6.1 Excavation--The first cell was excavated to a maximum depth of 6 feet. This provides a minimum of 5 feet separation between the lowest layer of waste and the highest seasonal groundwater elevation. The sidewalls were excavated to a maximum slope of 3:1. The bottom has a 1-percent slope to the center of the landfill.

The earthen material removed during excavation was segregated and stockpiled for future use as cover material and waste stabilizer.

TABLE 7-5. HEAVY METAL CONTENT AND PERCENT SOLIDS OF LIME SLUDGE DISPOSED AT SITE M

Sample date	Cadmium (mg/kg)	Chromium (mg/kg)	Copper (mg/kg)	Cyanide (mg/kg)	Iron (mg/kg)	Lead (mg/kg)	Nickel (mg/kg)	Zinc (mg/kg)	pH	Total solids (%)
7/79	6.6	200	4,500	750	5,400	5,200	1,300	5,700	7.8	--
7/80	1.6	110	5,300	140	10,000	5,100	1,100	4,900	8.6	11.1
10/81	3.3	1,300	3,800	2	11,000	4,800	340	5,200	7.9	28.7
7/82	0.89	170	520	52	880	520	64	640	8.5	12.1

7.2.14.6.2 Subsurface Monitoring System--Four trenches were excavated in the landfill base to a depth of 2 feet. Four-inch perforated PVC pipes and coarse sand and gravel were placed in these trenches to serve as the subsurface monitoring system.

7.2.14.6.3 Clay Liner--Prior to the placement of the clay liner, all large rocks, roots, and other foreign matter were removed from the foundation. The foundation was then graded to a 1-percent slope and scarified to permit better bonding between the foundation and the first clay lift. Uncompacted liner material was placed in loose 12-inch lifts and moistened as necessary. The clay was then compacted with "approved" equipment. Documentation of the specific types of equipment used was not available.

7.2.14.6.4 Leachate Collection System--The leachate collection system consists of 4-inch perforated PVC pipes placed in a 12-inch layer of sand on top of the clay liner. The 4-inch pipes run into a 6-inch pipe in the center of the landfill, which leads to a 1,000-gallon concrete tank and manhole system. Documentation of the construction procedures used to install the leachate collection system was not available.

7.2.14.7 Performance--

Samples from the monitoring wells have been analyzed on a quarterly basis since 1978. Examples of these data are presented in Table 7-6. These data indicate that the groundwater quality has not changed significantly since the commencement of the landfill operation.

The leachate from the landfill has also been sampled and analyzed since 1978. Examples of these data appear in Table 7-7. Table 7-8 contains leachate volume data.

The subsurface monitoring system tank also has been checked on a quarterly basis. There has been no indication of leachate entering this system.

The above facts confirm that the landfill has performed as designed since its construction and initial waste placement in 1977 and 1978.

7.2.15 Site N

7.2.15.1 Physical Description--

The site is located in an old sand and gravel pit and covers approximately 160 acres. The eight cells range in size from 9 acres to 25 acres with 20 acres being the average cell size. At the time of writing, the liner covers 108 acres.

The facility bottom is lined with a 4-inch bentonite and sand layer. The sides of the landfill are unlined. Two 1,000-ft^2 PVC-lined lysimeters are below the liner as well as a system of 24-inch perforated pipe, which lowers the groundwater table. A leachate collection system is above the liner. In addition to these systems, several monitoring wells and surface-water monitoring points are around the facility (see Figures 7-20 and 7-21).

TABLE 7-6. GROUNDWATER MONITORING WELL SAMPLE ANALYSIS AT SITE M

Well number	Sample date	Cadmium (mg/kg)	Chromium (mg/kg)	Copper (mg/kg)	Cyanide (mg/kg)	Iron (mg/kg)	Lead (mg/kg)	Nickel (mg/kg)	Zinc (mg/kg)	pH
1	7-79	<0.01	<0.05	<0.05	<0.01	0.30	<0.1	<0.05	0.01	8.0
1	7-82	<0.01	<0.05	<0.05	<0.02	0.10	<0.2	<0.05	0.5	7.5
2	7-79	<0.01	<0.05	<0.05	<0.01	0.25	<0.1	<0.05	0.15	7.9
2	7-82	<0.01	<0.05	<0.05	<0.02	0.4	<0.2	<0.05	0.10	7.2
3	7-79	<0.01	<0.05	<0.05	<0.01	0.1	<0.1	<0.05	<0.01	7.9
3	7-82	<0.01	<0.05	<0.05	<0.02	0.05	<0.2	<0.05	0.02	7.3

TABLE 7-7. LEACHATE ANALYSIS AT SITE M

Sample date	Cadmium (mg/L)	Chromium (mg/L)	Copper (mg/L)	Cyanide (mg/L)	Iron (mg/L)	Lead (mg/L)	Nickel (mg/L)	Zinc (mg/L)	pH
7/79	<0.01	<0.05	0.15	0.02	3.5	<0.1	0.10	0.13	7.4
7/80	<0.01	<0.05	0.20	0.03	4.0	<0.1	0.05	0.10	7.5
10/81	<0.01	<0.05	0.30	0.09	3.0	0.1	0.35	0.10	6.9
7/82	0.02	0.05	0.35	<0.02	3.5	<0.2	0.15	<0.10	7.2

TABLE 7-8. LEACHATE VOLUMES AT SITE M

Date	Volume
1/82	31,000
4/82	150,500
7/82	124,520
10/82	66,925

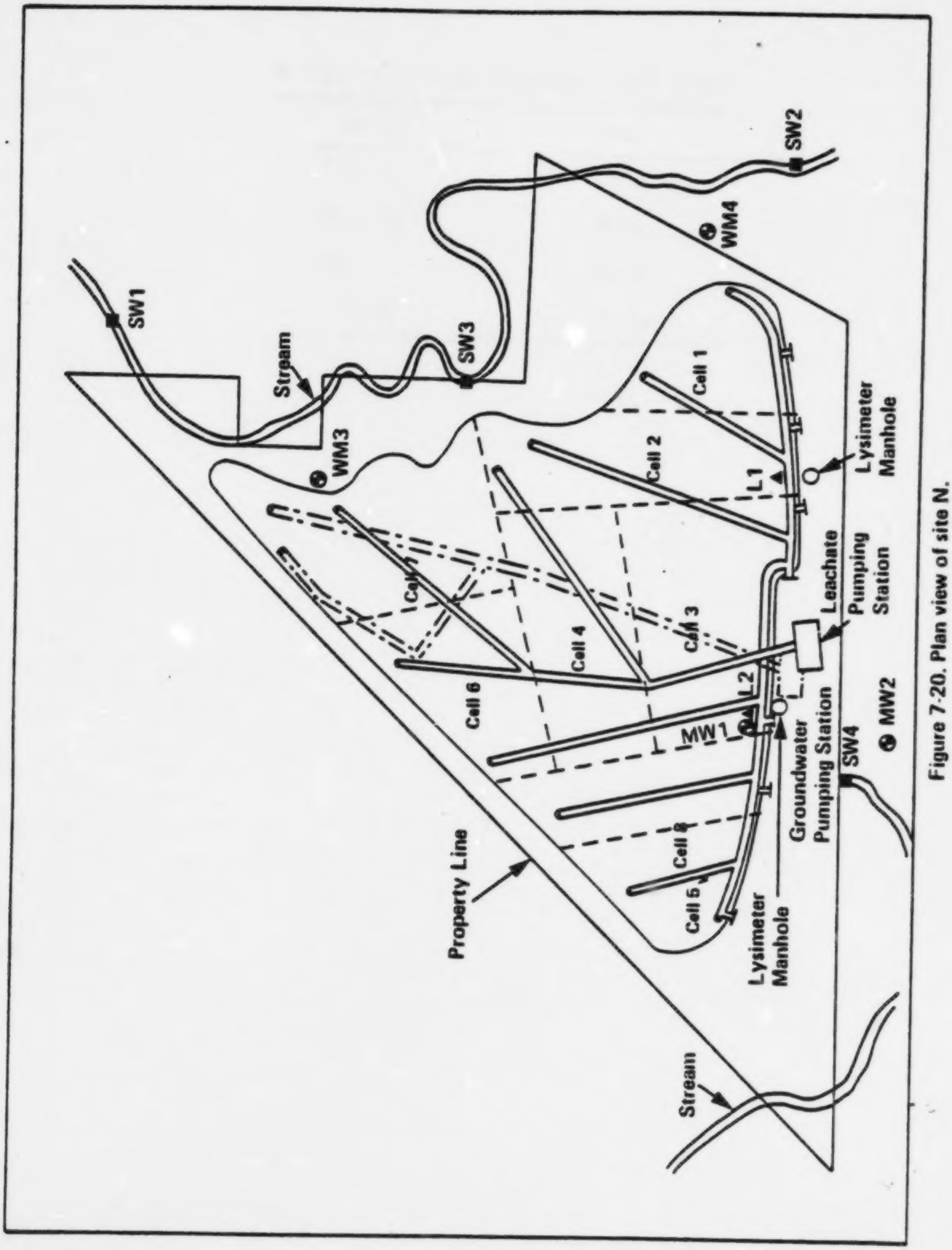

Figure 7-20. Plan view of site N.

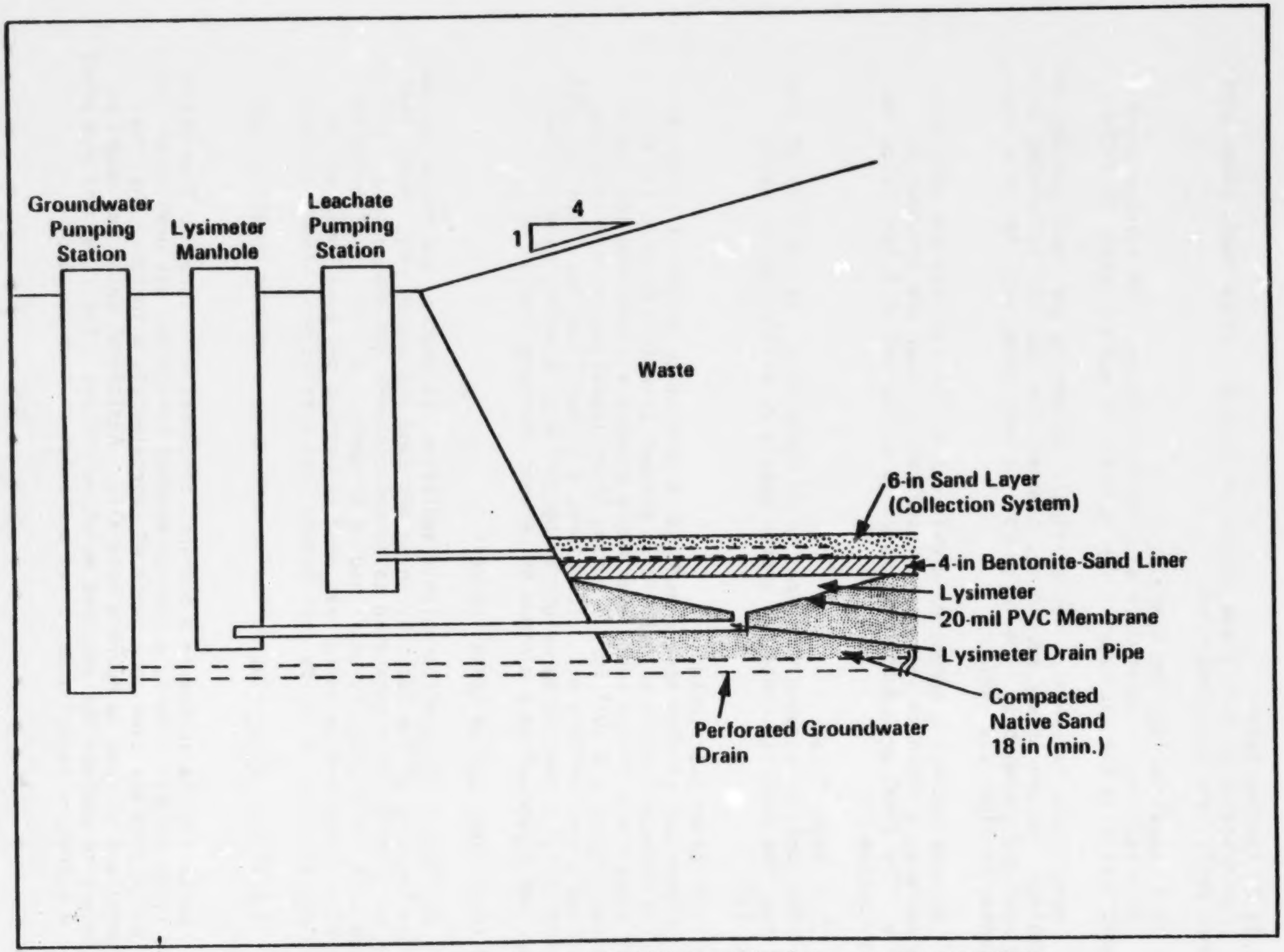
Groundwater Pumping Station
Lysimeter Manhole
Leachate Pumping Station
1
4
Waste
6-in Sand Layer (Collection System)
4-in Bentonite-Sand Liner
Lysimeter
20-mil PVC Membrane
Lysimeter Drain Pipe
Compacted Native Sand 18 in (min.)
Perforated Groundwater Drain

Figure 7-21. Cross-sectional view of site N liner and leachate management systems.

7.2.15.2 Startup Date--

Construction of cell 1 was initiated in 1974. Since then, seven additional cells have been installed.

7.2.15.3 Local Geology and Hydrology--

This facility is located in southeastern Canada. The average annual precipitation in the vicinity of the landfill is approximately 35 inches.

Very little information was available concerning the local geology and hydrology. As previously mentioned, however, the facility is located in an old sand and gravel pit. The local soil is very sandy and, for this reason, a bentonite liner was chosen.

Because natural groundwater elevations at the facility are very near the surface, a drainage system was installed to lower the groundwater table. The final groundwater elevation is maintained at 5 feet below the liner bottom.

7.2.15.4 Waste Type--

The facility accepts municipal solid waste only. At the time of this writing, the facility contained approximately 25 million yd^3 of waste material.

7.2.15.5 Liner Description--

Laboratory studies have shown that 6 percent by weight of bentonite (sodium montmorillonite with 55 to 75 percent by weight passing the #70 mesh sieve) in a 6-inch sand layer would produce a liner with the specific permeability of 1×10^{-8} cm/s. During liner installation, a 4-inch layer produced a more uniform mix. Therefore, a 4-inch liner was installed with an additional 3 percent by weight of bentonite as a safety margin. The liner was compacted to a minimum 90 percent standard Proctor density.

7.2.15.6 Liner System Installation--

7.2.15.6.1 Excavation--Little additional excavation was necessary due to the location of the facility in an old sand and gravel pit. Areas that were to be lined were excavated to bottom contours and graded to a 1-percent slope with large-capacity self-loading scrapers. A 7-ton, self-propelled, vibratory, smooth-drum roller was used to compact the basal sand and to complete the grading of the cell bottom into a relatively smooth surface.

7.2.15.6.2 Liner--The following methods were used to construct the liner.

Bentonite was spread to a 5/8-inch thickness with three to five passes of a large scraper. Next, a tractor-mounted rotatiller was used to mix the bentonite into the sand to a depth of approximately 4 inches. The liner material was allowed to hydrate naturally. Additional water was added as necessary to achieve the required moisture content. The liner was compacted with a vibratory smooth-drum roller a minimum of four passes.

The leachate collection system was then installed on the liner and covered with clean pea gravel. A 6- to 8-inch layer of loose sand was placed over the finished liner. No traffic was allowed on the completed area before refuse was spread over the liner from the top.

QA testing of the constructed liner indicated that the bentonite content ranged from 8.5 to 15 percent by weight and averaged about 11.2 percent by weight. Greater than 90 percent compaction was achieved in all areas with variations from 93 to 100+ percent standard Proctor density. The moisture content varied from 3 to 12 percent.

7.2.15.6.3 Monitoring Systems--Methods used to install the various monitoring systems were not available.

7.2.15.7 Performance--

Analysis of water samples collected from lysimeter 1, surface-water monitoring points, and groundwater monitoring wells indicates that leachate has passed through the liner and entered the groundwater. Liquids have also been detected but not analyzed in lysimeter 2. Table 7-9 contains the results of the analysis for biochemical oxygen demand (BOD), COD, total coliform, and fecal coliform at several of the monitoring points.

7.2.16 Site O

7.2.16.1 Physical Description--

This landfill is approximately 2 acres in size. It is lined with a mixture of local soil combined with a 1:1 mix of bentonite and lime. The lime was added to reduce the swelling potential of the bentonite. A leachate collection system is on top of the clay liner. Several groundwater monitoring wells are located adjacent to the landfill. The landfill has been capped and covered with asphalt pavement.

The leachate collection system consists of a stone-packed sump (drywell) with a casing pipe extending to the surface that is used for observation and leachate withdrawal.

7.2.16.2 Startup Date--

Construction of this facility began in June of 1980. It was filled and capped by September 1980.

7.2.16.3 Local Geology and Hydrology--

This facility is located in the southeastern United States. Average annual precipitation at the facility is approximately 50 inches.

The soil in the vicinity of the landfill is composed of a mixture of clay, silt, and sand. Soil borings of the site indicate that this soil mixture extends to a minimum of 10 feet. Analysis of the native soil gave the following results:

TABLE 7-9. WATER SAMPLE ANALYSES: BOD, COD, TOTAL COLIFORM, AND FECAL COLIFORM

Sampling location (see Figure 7-20)	BOD (mg/L)	COD (mg/L)	Total coliform 100 mL	Fecal coliform 100 mL
L 1[a]	336	1,560	14,000	14,000
SW 1[b]	<1	3.9	500	140
SW 2[b]	<1	5.8	1,900	320
SW 3[b]	1	19	500	90
SW 4[b]	260	400	0	0
MW 1[b]	>800	1,870	0	0
MW 2[b]	16	45	190	0
MW 3[b]	13	100	0	0
MW 4[b]	2	10	0	0

[a] Sample date - 12/5/83.

[b] Sample date - 10/11/83.

Average permeability	8.7×10^{-5} cm/s
Plasticity index	7.1%
Liquid limit	27.1%
Plastic limit	20.0%
Amount passing No. 200 sieve	31.3%.

Monitoring wells have indicated that groundwater is well below the bottom of the site and poses no potential problem. Water samples from the monitoring wells are taken monthly and analyzed for PCB's, pH, specific conductance, and chlorinated organics.

7.2.16.4 Waste Type--

The major material disposed at this site was PCB's. The facility also contains solvents, waxes, and oils, all of which were solidified with sawdust prior to their disposal.

7.2.16.5 Liner Description--

The 4-foot-thick liner consists of a homogeneous mixture of 3 percent bentonite, 3 percent lime, and 94 percent native soil.

The physical properties of the liner material are:

Liquid limit	27.8%
Plastic limit	20.7%
Plasticity index	7.1%
Permeability	8.3×10^{-8} cm/s
Amount passing No. 200 sieve	35.4.

The leachate collection system consists of a stone-packed sump (drywell) with a casing pipe extending to the surface that is used for observation and leachate withdrawal.

7.2.16.6 Liner Installation--

The liner at this facility was installed in 8-inch lifts and compacted to at least 95 percent of maximum Proctor density. The bottom of the facility slopes at a rate of 2 percent to the leachate collection system. The sidewall slopes are 3:1 maximum. The 3-foot-thick cap constructed of the same mixture was compacted to 90 percent of maximum density. No information on the installation of the asphalt cover was available.

7.2.16.7 Performance--

Due to a poor seal around the leachate collection system casing pipe (standpipe), surface water eroded through this area and was collected in the sump. This problem was solved by removing a 12-foot-diameter section of the cap surrounding the pipe. The area was then filled with tightly compacted pure bentonite. A concrete dome was placed on top of the bentonite to direct the flow of water away from the casing pipe. Because additional liquids were still being collected in the collection system, the entire cap was paved with asphalt. No information was available on the performance of the facility after the asphalt was installed.

7.2.17 Site P

7.2.17.1 Physical Description--

The facility consists of one double-lined hazardous waste cell. The bottom liner is composed of leachate collection and detection systems as well as a series of natural soil and bentonite/soil liners. The side liner and dike-containment system include a soil and a bentonite/soil liner (see Figures 7-22 and 7-23).

7.2.17.2 Startup Date--

The facility was constructed and began accepting waste in 1980.

7.2.17.3 Local Geology and Hydrology--

This facility is located in the southeastern United States. The average annual precipitation in the vicinity of this facility is approximately 52 inches. No information on the local geology was available.

Groundwater at this site occurs in soil and fractured rock. The maximum groundwater table is located approximately 14 to 15 feet below the liner bottom. Groundwater levels at the site fluctuate with precipitation, tending to be high in winter and spring and low in the remainder of the year.

7.2.17.4 Waste Type--

Approximately 80 percent of the disposed waste is brine purification mud. This material has the consistency and appearance of fine, wet sand. The remaining 20 percent of the wastes are all mercury-contaminated substances including sulfide treatment filter cake, retort ash, Reductone® filters, decomposer packing, and contaminated earth.

7.2.17.5 Liner Description--

The liner at this facility is composed of two sections: a bottom liner and a side (dike) liner. Each of these sections has its own specifications.

The bottom liner extends over the entire bottom and 6 feet up the sides. It is a layered system containing two drainage or collection layers and two soil liners. From the top layer downward, the bottom liner components are as follows:

- Leachate collection system--1 foot of #78 gravel and sand with 4-inch perforated PVC pipes.

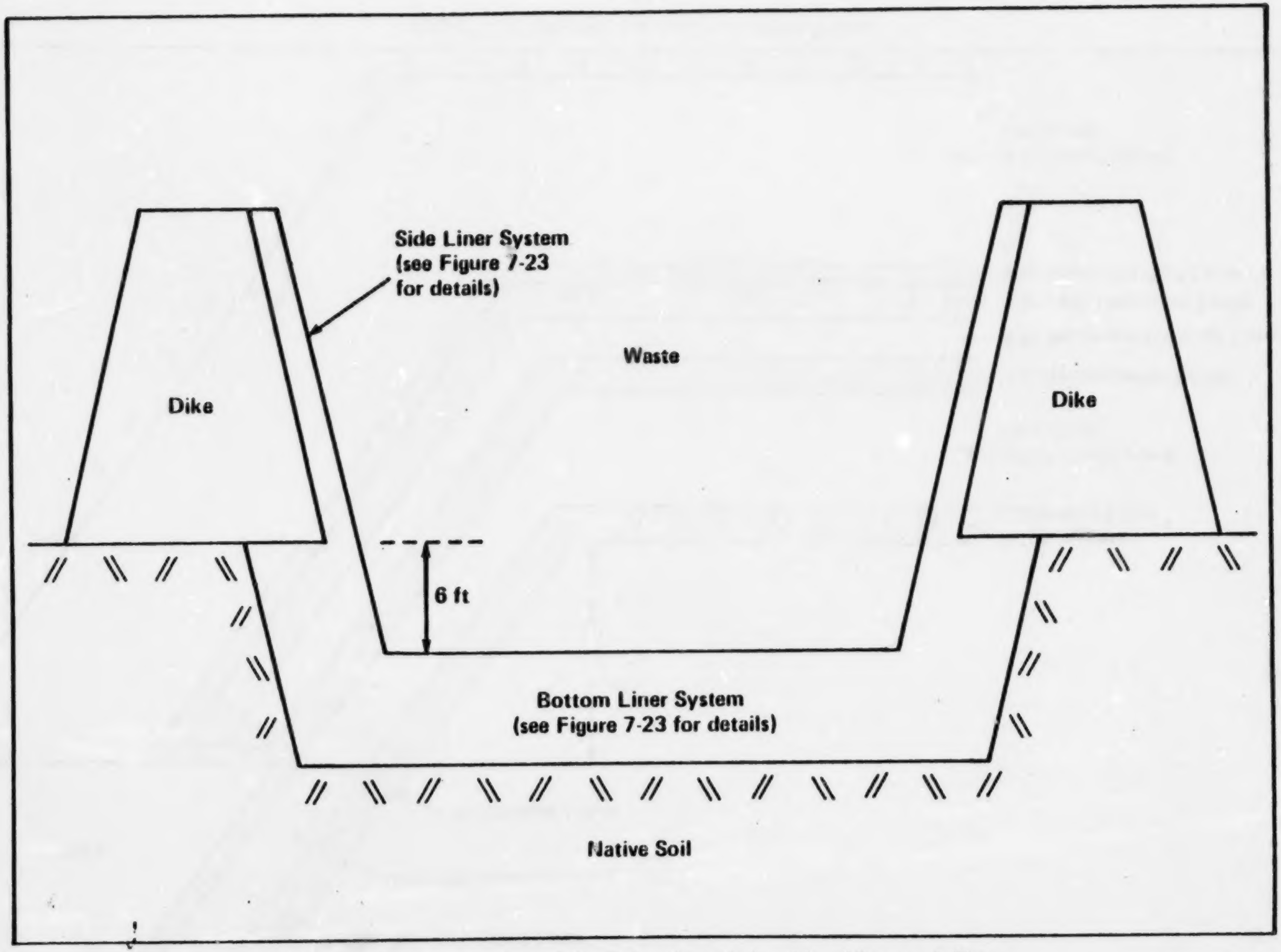

Figure 7-22. Cross section of site P showing relationship of liner and dikes.

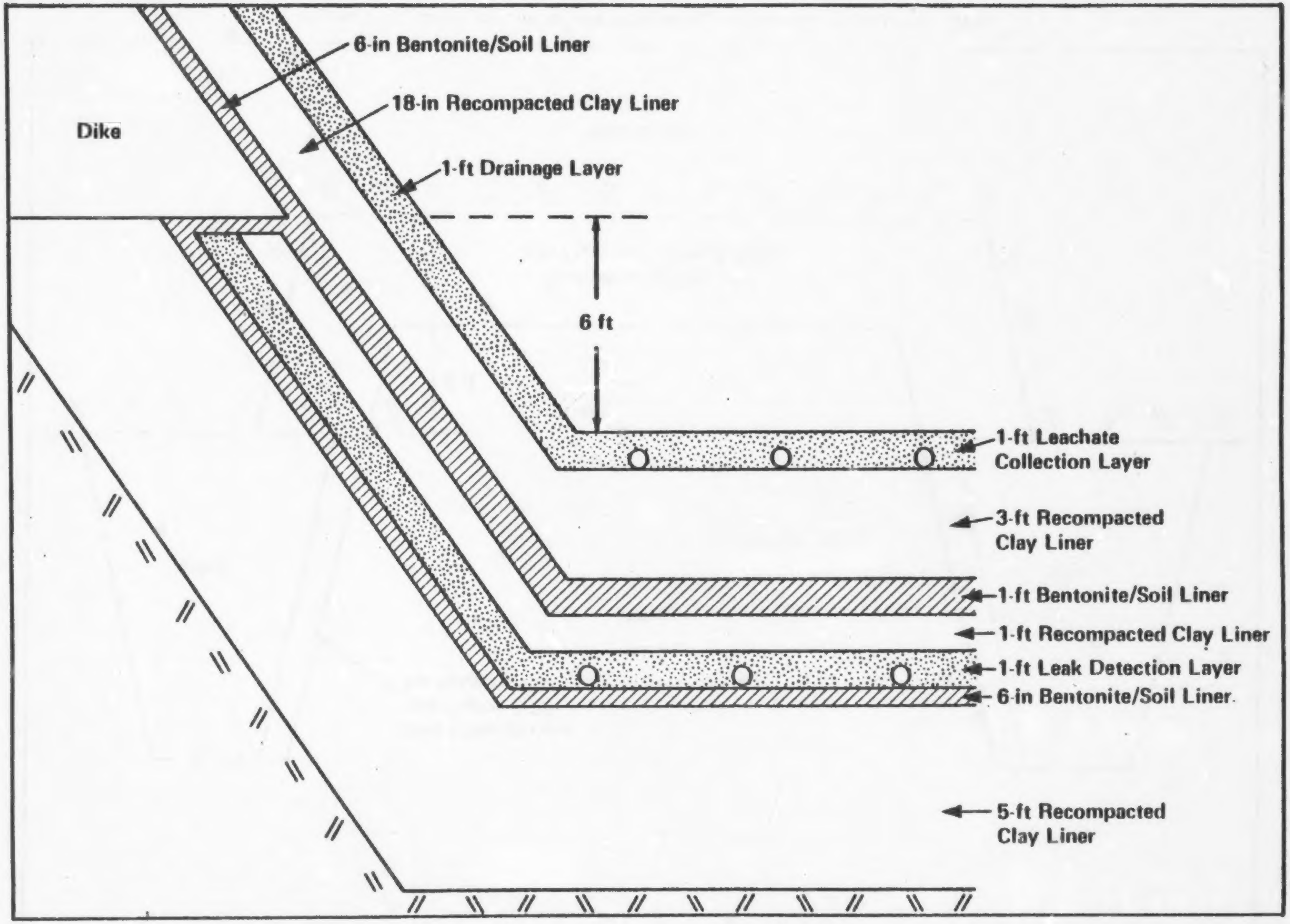
6-in Bentonite/Soil Liner
18-in Recompacted Clay Liner
Dike
1-ft Drainage Layer
6 ft
1-ft Leachate Collection Layer
3-ft Recompacted Clay Liner
1-ft Bentonite/Soil Liner
1-ft Recompacted Clay Liner
1-ft Leak Detection Layer
6-in Bentonite/Soil Liner.
5-ft Recompacted Clay Liner

Figure 7-23. Detailed cross section of site P liner.

- Upper soil barrier--5 feet of compacted soil further subdivided into three layers:
    - 3 feet of compacted native soil; permeability = 1×10^{-4} cm/s.
    - 1 foot of enhanced soil, i.e., native soil blended with 9 to 12 percent polymer-treated bentonite; permeability on the order of 5×10^{-8} cm/s.
    - 1 foot of compacted native soil; permeability = 1×10^{-4} cm/s.
- Leak detection layer. A 1-foot sand/gravel layer with perforated pipe, to detect leaks and/or to control seepage through the upper barrier. The pipes are connected to several independent monitoring stations to allow the determination of the approximate location of any leaking that might occur.
- Lower soil barrier. A 6-inch layer of enhanced soil, i.e., native soil blended with 9 to 12 percent polymer-treated bentonite; permeability on the order of 5×10^{-8} cm/s.
- Buffer zone. 5 feet of either in situ or recompacted native soil. Permeability = 1×10^{-4} cm/s.

The side liner system extends from a point 6 feet above the cell bottom to the top of the cell. This section will not have liquid impounded against it; therefore, the liner system is not as extensive as the bottom liner. From the top layer downward, the side liner components are as follows:

- 1 foot of #78 gravel
- 18 inches of compacted native soil; Permeability = 1×10^{-4} cm/s
- 6 inches of enhanced soil, i.e., native soil blended with 9 to 12 percent polymer-treated bentonite.

Liner-waste compatability tests were conducted from June through December 1980 with a percolation column. In it was placed a compacted blend of 1,600 g native soil and 360 g bentonite (i.e., 18.4 percent bentonite). The column was filled with actual facility liquids, with analysis as follows:

- 1.87 ppm mercury
- 154,036 ppm chlorides
- pH 6.1.

Short-term permeability results are as follows:

Date	Permeability (cm/s)*
6-2-80	2.23×10^{-7}
6-10-80	2.92×10^{-7}
6-17-80	3.71×10^{-7}
7-2-80	4.05×10^{-7}
7-18-80	4.81×10^{-7}
7-24-80	5.88×10^{-7}
Ca. 12-80	Approximately 0

7.2.17.6 Liner System Installation--
The construction methods used are as follows:

- Soil spreading. All soils and gravel were placed with scrapers and spread with bulldozers.

- Soil compaction. All soils were compacted to 95 percent Standard Proctor density. This was done by repeated front-end loader or scraper passes (rubber-tired). An independent soils laboratory took samples as the work progressed, and any fill not meeting the minimum 95 percent compaction was upgraded accordingly.

- Mixing of soils and bentonite. The bottom 6-inch layer of "enhanced" soil was constructed by first spreading two lifts of approximately 4.5 to 5 inches of loose native soil. Likewise, the top 12-inch layer of "enhanced" soil was constructed by first spreading four such lifts. Scrapers were used to spread the soil. After placement of loose soil for each layer, the bentonite product was hand-placed at an application rate of 50 lb/40 ft^2. Mixing was accomplished with a rotivator until the color of the bentonite became unnoticeable.

- "Enhanced" soil compaction was done by repeated passes of a front-end loader or rubber-tired scraper until a minimum of 95 percent Proctor was achieved.

- Quality assurance. Soil layers were sampled as the work progressed to ensure 95 percent standard Proctor density. During construction of the "enhanced" soil layers, a minimum of one field density test was performed per 2,500 ft^2 of lift. Tests for moisture, permeability, grain-size distribution, and liquid and plastic limits were also performed by qualified soils personnel on an "as-needed" basis.

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*Calculated values based on leachate absorbed into test specimen. All data supplied by American Colloid Company.

Results from the quality assurance of the constructed liner indicate that the wet unit weight ranged from 101 lb/ft^3 to 115 lb/ft^3, the moisture content ranged from 23 to 28 percent, and the laboratory permeability ranged from 2×10^8 cm/s to 7×10^8 cm/s.

7.2.17.7 Performance--

Leachate collected in the leachate collection layer was analyzed and found to contain high levels of chlorides and measurable levels of mercury. Small amounts of liquid have also been collected from the leak detection system. This liquid does not contain high levels of either chlorides or mercury. The monitoring wells have not shown any significant changes.

The performance of this facility is very difficult to determine at this time. A complete analysis of the liquid collected in the leak detection system is being performed. Attempts are being made to correlate these results with changes in precipitation, waste type, and water table level. The complete analysis was not available at the time of this writing.

7.2.18 Site Q

7.2.18.1 Physical Description--

The landfill consists of a single containment cell covering an area of approximately 3 acres. It is located in a former sand and gravel pit. The facility has a double liner consisting of two 4-inch bentonite/soil layers. These layers are separated by a leak detection system.

7.2.18.2 Startup Date--

This facility was placed in operation in October 1976.

7.2.18.3 Local Geology and Hydrology--

This facility is located in the northeastern United States. Average annual precipitation at this facility is approximately 35 inches.

The immediate area around the facility is mostly sand and gravel. The facility is located above the 100-year flood plain.

7.2.18.4 Waste Type--

The sludge deposited in the landfill is a black, semisolid, relatively odorless material, which will support the weight of small grading equipment. The sludge is nonvolatile and noncombustible. It consists primarily of metal hydroxides and waste pigments. Typical sludge composition is as follows:

Metal	Percentage
Iron	10 - 24
Lead	5 - 20
Chromium	6 - 12
Barium	2 - 12
Calcium	4 - 48
Aluminum	1 - 3
Copper	1 - 2
Cadmium	1 - 2
Zinc	0 - 1
Carbon	2 - 10

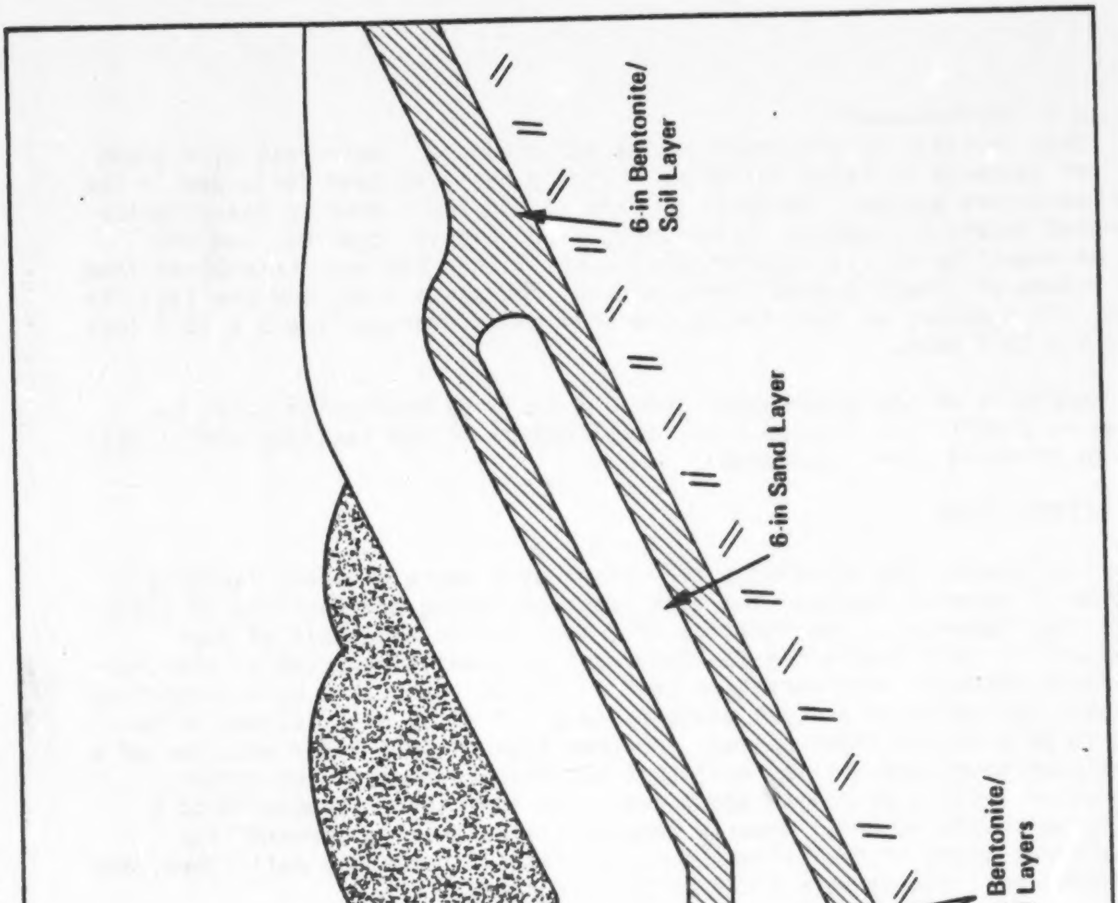
7.2.18.5 Liner Description--

The cell has a double liner consisting of two 4-inch layers of a bentonite/soil mixture on the bottom and side slopes up to a vertical elevation of 20 feet above the cell bottom. The bentonite/soil layers are separated by a 12-inch layer of sand on the bottom of the cell and a 6-inch layer of sand on the side slopes. The side slopes above the 20-foot vertical level are covered with a single 6-inch layer of the bentonite/soil mixture. All bottom and side slope bentonite/soil surfaces are covered with a 12-inch protective layer of gravel. The slope of the cell sidewalls varies from 2.5:1 to 3:1. A typical cross section of this facility is illustrated in Figure 7-24.

7.2.18.6 Liner System Installation--

Most of the bentonite/soil mixture used for the liner was premixed in a pug mill to predetermined proportions by closely controlling the feed rate of each material. Soil was fed from a hopper onto a conveyer belt at a constant rate. Dry bentonite was fed from a hopper onto the conveyor belt on top of the soil through a variable-rate vibratory feeder. The feed rate of the bentonite was periodically checked and adjusted as necessary by weighing timed samples. The two materials were discharged into the pug mill where water was added at a controlled rate to produce a thoroughly mixed product at desired moisture content (specific value unknown). The mixture was discharged from the pug mill into dump trucks and was placed in position at a predetermined thickness with a grader, screened boards, and hand labor. The mixture was then compacted to the specifications with a backhoe-mounted hydraulic tamper.

During all construction phases, CQA and CQC required full-time supervision by qualified personnel and a soils consultant as required.



Waste
12-in Gravel Layer (Leachate Collection)
12-in Sand Layer (Leak Detection)
4-in Perforated Pipes
4-in Bentonite/ Soil Layers
6-in Sand Layer
6-in Bentonite/ Soil Layer

Figure 7-24. Cross section of site Q liner.

7.2.18.7 Performance--

This facility is presently active and uncapped. As a result, a great deal of leachate is being collected. Liquid has also been collected in the leak detection system. Analysis of this liquid shows that it has slightly elevated levels of cadmium, lead, zinc, iron, copper, cyanide, and COD. The permeability of the upper bentonite/soil liner has been determined from the volume of liquid pumped from the sump, the waste head, and the facility area. This value, as reported by the site owner, varies from 3×10^{-8} cm/s to 6.5×10^{-8} cm/s.

Analysis of the groundwater from the facility monitoring wells has shown no significant changes since construction of the facility and is well within drinking water standards.

7.3 LINER TYPES

The feasibility of using a clay liner at a waste disposal facility depends on several factors, the most important being availability of suitable liner material. The location of a facility in a deposit of low-permeability soil lowers the facility cost by greatly reducing or eliminating liner material transportation costs. The cost incurred by transporting suitable borrow soils a short distance (e.g., 5 miles or less) may allow this to be a viable liner option. Another type of soil liner consists of a relatively high-permeability soil that has been augmented with certain natural or treated bentonite additives. The addition of bentonite to a highly permeable material greatly reduces the material's permeability. These three types of facilities (i.e., unlined, recompacted soil lined, and bentonite/soil lined) were included in this study.

Two of the four unlined facilities discussed have had groundwater contamination problems. One facility had a problem due to a faulty well installation. No other problems have been reported at this facility. The fourth unlined facility has had contamination detected in one of its monitoring wells. A study of this contamination indicated that its source was an adjacent drum disposal facility.

Nine facilities with recompacted soil liners are discussed in this chapter. Three of these nine facilities have had some type of performance problem. These three facilities all have leak detection systems that were responsible for quickly detecting and providing the information necessary for determining the cause of the failures.

Four facilities with admixed (bentonite/soil) liners are discussed in this chapter. Two of these facilities have had small amounts of liquid detected in between their double liners. However, overall they appear to be functioning according to their design specifications. One facility had a cap problem that was rapidly detected and corrected. The liner at this facility has had no detected problems. The final facility with a bentonite/soil liner that is discussed in this section has had performance problems. Leachate has migrated through the 4-inch liner and has resulted in extensive groundwater contamination.

7.3.1 Unlined Facilities

Although the purpose of this document is to discuss clay liners, in some cases information relating to waste disposal and clay liners may be obtained from the case histories of unlined disposal sites. Generally speaking, unlined sites are located in soil formations that are relatively low in permeability. However, such soil formations are rarely homogeneous. They generally contain sand or gravel seams, varying amounts of organic matter, and cracks or fissures. In some cases, they may also contain lenses of gypsum, limestone, or other soluble material. Discontinuities such as these are often excavated and then backfilled with recompacted clay prior to waste disposal. When this is not specified or when highly permeable areas are unnoticed during facility construction, the siting of an unlined hazardous waste disposal facility in a heterogeneous soil may cause severe performance problems.

Examples of unlined waste disposal facilities are sites A, B, C, and G. These four facilities are 8 to 30 years old. Extensive groundwater contamination has occurred at two of these facilities: a slight problem due to poor well installation procedures occurred at one facility, and contamination in a well at the fourth facility is thought to have come from an adjacent abandoned drum disposal operation.

Facility B, a zone-of-saturation or intergradient landfill, has had severe groundwater contamination problems. This facility is located in a low-permeability glacial till deposit. Lenses of sand or gravel that were encountered during the site construction were excavated and backfilled with a minimum of 5 feet of recompacted clay. A buildup of leachate in the landfill due to the disposal of unsolidified liquids, precipitation, and seepage caused waste to enter a permeable deposit that had been improperly sealed with clay. This resulted in leachate migration and groundwater contamination.

Facility C is also unlined. Geological investigations revealed that calcium carbonate seams and nodules were present throughout the entire area. The design specifications called for the excavations of such seams and then required the excavated areas to be backfilled with the local clay-shale soil. Groundwater contamination at this facility can be attributed to inadequate excavation of the calcium carbonate deposits, inadequate recompaction of the excavated areas, or lack of sufficient waste/liner material compatibility testing.

Initial groundwater samples taken from a monitoring well at site A had pH values between 10.7 and 11.3. An investigation into the problem revealed that waste migration from the unlined facility was not the cause. Instead, the contamination was traced to some steel-mill slag that was used as well packing instead of the specified clean gravel. This problem was remedied, and since that time no other problems have been reported.

Facility G is an old, unlined hazardous waste disposal facility. Samples from a groundwater monitoring well at this facility have contained high levels of various pollutants. A study of this contamination indicated that its source was an old drum disposal site (previously owned by another company) located adjacent to the landfill. This drum disposal site is located on a sand and gravel seam. This permeable seam was determined to be the mechanism by which the contaminants were transported to the monitoring well at Site G.

7.3.2 Recompacted Soil Liners

Recompacted soil liners are the most common type of soil liner in current use. This category consists of both recompacted in situ soil liners and liners constructed of material that has been transported to the facility from a nearby borrow area. Sites D, E, F, H, I, J, K, L, and M are all in this category. Some of these facilities have a double liner system (in the case of sites D and F, one of the liners is a synthetic). When this is the case, the drainage layer separating the clay liners is called a leak detection layer or system. All of these facilities have either a leachate collection or a leak detection system (in some cases, both). These facilities are 4 to 9 years old. The performance at these facilities is quite varied. Six of the facilities (sites D, E, F, H, K, and M) appear to be functioning as planned with only a few minor problems. In the case of site K, the liquid volumes removed from the detection system were used to calculate the liner permeability. The calculated values were all lower than the design specifications required.

The remaining three facilities (sites I, J, and L) have all had performance problems. For example, the clay liner for a pond at site I was left empty and unprotected for several months. During this time desiccation cracks formed in the liner. Failure to repair the liner prior to waste placement resulted in waste migration into the leak detection layer. Waste migration into the leak detection system also occurred at site L shortly after waste placement. An investigation was unsuccessful in determining the location of the leak. However, it was determined that the lower compacted clay liner was not leaking. The other recompacted clay-lined facility with performance problems, site J, also has a leak detection system. In this case, perchloroethylene was mistakenly placed into a pond. Approximately 6 to 9 months later, and literally overnight, the "perc" penetrated the liner and was detected in the leak detection system. The apparent incompatibility of the "perc" and the liner material is consistent with current research on the effect of chlorinated solvents on clays.

7.3.3 Admixed Liners

Admixed liners are those composed of permeable soil and an additive designed to decrease its permeability. Commonly used additives include, asphalt, fly ash, soil cement, and polymer-treated or natural bentonite. Bentonite, a natural clay mineral, is the only additive discussed in this document.

The addition of bentonite (generally 3 to 15 percent is used) to a highly permeable soil can decrease its permeability so that it can be used as a liner material. A discussion of the physical and chemical properties of bentonite as well as its advantages and disadvantages as a liner material may be found in Chapters 2 and 5 of this document.

Facilities N, O, P, and Q are all lined with bentonite/soil admixtures. These facilities are 5 to 11 years old. The liner configurations at these facilities are quite varied. Site O has a 4-foot-thick native soil liner containing 3 percent bentonite and 3 percent lime. The lime was added to reduce the swelling potential of the bentonite. Sites P and Q are lined with a layered system containing bentonite/soil liners ranging from 4 to 12 inches thick and leak detection systems. Site P has the additional protection of a series of compacted native soil liners. Facility N is lined with a 4-inch bentonite/sand liner. Two lysimeters and a groundwater pumping system are located below the liner, and a leachate collection system is on top of the liner.

Just as the liner systems at these four facilities are varied, so are their performances. Both sites P and Q have had small amounts of liquid detected in their leak detection layers; however, overall they seem to be functioning according to specifications. In the case of site Q, the liquid volumes collected were used to calculate the as-built liner permeability. This value ranged from 3×10^{-8} cm/s to 6.5×10^{-8} cm/s, which is less than the specifications required. The source and quality of the liquid being collected from the detection system at site P are currently being investigated by the site owner.

Site O had a problem with liquid eroding the cap and infiltrating through the eroded section. This problem was corrected by repairing the eroded portion and then paving the entire cap with asphalt. The liner at this facility has not had any detectable problems.

Finally, facility N, a very large municipal waste disposal facility located in an abandoned sand and gravel pit, has produced extensive contamination that has been detected in the subsurface monitoring layer, the lysimeters, and several groundwater monitoring wells. Analysis of the contamination indicates that the leachate is passing through the 4-inch liner and entering the groundwater.

7.4 SITE CHARACTERIZATION

The location of a clay-lined hazardous waste disposal facility may have a significant effect on its performance. The site-specific geology, hydrology, and climate must be evaluated thoroughly prior to the facility design and construction. Some of the geological factors that must be investigated include the local soil type, permeability, and nature of the soil deposit (e.g., whether it contains continuous or discontinuous sand or gravel seams, gypsum or other reactive material seams and the depth of the deposits).

Hydrological factors that must be investigated include location of the water table, location of any perched water tables above the main water table, and the groundwater and surface-water flow patterns. Problems associated with high water tables include excessive pressure on the side or bottom liners. This hydraulic pressure may cause sidewall slumping or collapse or bottom heaving. High water tables may also result in the unwanted infiltration of groundwater. For a facility under construction, this may make compaction and other liner installation procedures difficult or impossible without continuous groundwater pumping. For a closed facility, groundwater infiltration may result in overloading the leachate collection system, causing high leachate levels in the facility.

Site-specific climate data such as average temperature and amount of precipitation also must be known. Locations that have below-freezing temperatures for long periods of time have the potential for freeze/thaw cycling of unprotected liners. This may result in increased liner permeability and poor liner performance. Hot climates, on the other hand, may affect clay liners by causing excessive evaporation and possibly desiccation of the liner. Clay liners located in hot, arid climates are even more susceptible to desiccation and cracking. On the other hand, excessive precipitation at an open clay-lined facility may cause a buildup of hydraulic head on the liner, thus increasing the flow of liquid through it. Excessive precipitation may also cause a rise in the groundwater table and the previously discussed problems associated with this phenomenon.

A complete discussion of these site-specific factors, their associated problems, and the potential failure mechanisms involved may be found in the preceding chapters of this document.

7.4.1 Case Studies

Several performance problems or liner failures that may have been partially caused by inadequate site characterization are illustrated by case studies B, G, and I.

Site B, a zone-of-saturation or intergradient facility, is located in a low-permeability soil formation. Sand or gravel seams that were encountered during excavation were to be removed and replaced with recompacted clay. No additional recompaction was specified. A buildup of leachate within the liner due to precipitation, the disposal of free liquids, and seepage caused the gradient to reverse. Within the site, apparently a sand seam was either not discovered or not properly sealed during construction. The leachate flowed through this sand seam and caused severe groundwater contamination in a limited area of the property.

Another problem caused by a sand seam was discovered at site G. Contamination in one of the facility monitoring wells was initially attributed to waste migration from within the site. However, an investigation into the problem revealed that the source of contamination was an adjacent abandoned drum disposal facility. Leachate from this facility entered a water-bearing sand seam that encountered one of the wells at site G.

as a liner material. A discussion of the physical and chemical properties of bentonite as well as its advantages and disadvantages as a liner material may be found in Chapters 2 and 5 of this document.

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Just as the liner systems at these four facilities are varied, so are their performances. Both sites P and Q have had small amounts of liquid detected in their leak detection layers; however, overall they seem to be functioning according to specifications. In the case of site Q, the liquid volumes collected were used to calculate the as-built liner permeability. This value ranged from 3×10^{-8} cm/s to 6.5×10^{-8} cm/s, which is less than the specifications required. The source and quality of the liquid being collected from the detection system at site P are currently being investigated by the site owner.

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7.4 SITE CHARACTERIZATION

The location of a clay-lined hazardous waste disposal facility may have a significant effect on its performance. The site-specific geology, hydrology, and climate must be evaluated thoroughly prior to the facility design and construction. Some of the geological factors that must be investigated include the local soil type, permeability, and nature of the soil deposit (e.g., whether it contains continuous or discontinuous sand or gravel seams, gypsum, or other reactive material seams and the depth of the deposits).

Hydrological factors that must be investigated include location of the water table, location of any perched water tables above the main water

Site I is located in a semiarid climate. Three ponds with double clay liners were installed in the fall of 1980. Two of the ponds were filled shortly after construction, while the third pond was not filled until the following spring. During this time, the unprotected clay liner developed severe desiccation cracks. These were not repaired prior to filling; consequently, contaminated liquid passed through the liner and began accumulating in the leak detection system. This problem, which was caused by the arid climate and the loss of water from the liner, could have been avoided with adequate liner maintenance and a rigorous preservice inspection.

7.5 INSTALLATION OF CLAY LINERS

This section presents a very brief discussion of clay liner installation methodology and procedures. Only one case study is included for discussion because it is very difficult to attribute a liner failure to poor construction techniques. For a complete discussion of this topic, see Chapter 5.

7.5.1 Installation Methods

It is widely believed that the performance of clay liners is affected by the equipment and procedures used to install them. Conclusive field data to support this belief, however, are not available. Specific items that need to be controlled during clay liner construction are discussed in the following sections.

7.5.1.1 Excavating, Grading, and Foundation Preparation--

The excavation and grading of the bottom and side slopes of the liner foundation should be conducted according to design specifications. This generally requires the use of construction equipment such as dozers, backhoes, scrapers, and graders. For unlined facilities constructed in low-permeability soil, special care must be taken to locate and fully excavate all permeable zones that may allow for leachate migration. For facilities that are to be lined with recompacted local soil or admixed materials, suitable liner materials are generally excavated, stockpiled, and used as required. In all cases, the specified slopes and elevations of sidewalls and bottoms as well as those of collection system pipes and manholes must be carefully controlled. This will help ensure that the liner is properly located regarding the water table and local geological features. Specified side and bottom slopes are also necessary so that leachate will flow to the collection system pipes and sump, thus preventing the "bathtub" effect.

7.5.1.2 Liner Materials--

Native soil liner material may be excavated from the site during foundation preparation or may be brought to the site from a nearby borrow area. Admixture materials may be shipped to the facility in bags or bulk form. In all cases, the use of materials that are specified in the facility design is extremely important. As previously discussed, materials such as sand, rocks, roots, or other organic matter, if included in the liner, will greatly affect its permeability and ultimate performance. When materials such as bentonite are to be added to in situ materials to form a liner, the bentonite content must be carefully controlled. The use of inadequate quantities of bentonite will produce a liner with higher than required permeability and lower than specified performance.

7.5.1.3 Moisture Content/Density/Compactive Effort--

The relationship among moisture content, density, and compactive effort is discussed in Chapter 2. Careful control of these parameters is necessary throughout all phases of liner construction. The final liner permeability and performance depend on proper moisture control and distribution, mixing, and compaction techniques. Too little or too much water added to the liner material will make it difficult to compact to the specified density, thus affecting permeability. Inadequate mixing of materials may result in a heterogeneous liner. Insufficient compactive effort may result in a liner that is more permeable than required.

7.5.2 Quality Assurance/Quality Control For Clay Liners

A survey of hazardous waste surface impoundment technology has found that rigorous quality assurance is necessary to achieve good site performance (Ghassemi et al., 1984). Liner failures at several impoundments included in the survey were attributed to various factors including "failure to execute proper quality assurance and control." The success of surveyed facilities that have performed very well is attributed to many factors including "the use of competent design, construction and inspection contractors, close scrutiny of all phases of design, construction and QA inspection by the owner/operator, excellent CQA/CQC and recordkeeping during all phases of the project, and good communications between all parties involved in establishing the sites."

Specific problems that may result in clay liner failure and that can be avoided with careful CQA include:

- Use of materials with specifications other than the ones in the approved design
- Lack of careful screening and testing of incoming materials to remove roots and other organic matter, rocks, pockets of permeable materials, and other foreign objects prior to placement
- Lack of adequate moisture control both prior to and after compaction
- Improper size reduction, mixing, and spreading of liner materials
- Use of inadequate liner materials (especially important with bentonite/soil liners)
- Failure to follow installation procedures specified in the design

- Use of improper construction equipment
- Application or specification of inadequate compactive effort.

The following case study is of a surface impoundment that has had performance problems. Due to insufficient information, this case study is not included in Section 7.2. The problems at this facility may have resulted from a combination of factors, one being the lack of adequate CQA.

A sewage-treatment lagoon was to be lined with 3.7 pounds of polymer-treated bentonite per square foot of liner mixed with the native soil to a depth of 4 inches. The particle size of the bentonite that was supplied did not meet the original specifications, making it very difficult to spread and mix the material adequately. Problems were also encountered in achieving proper moisture content. After the bentonite/soil layer was compacted, the final permeability of the liner was to be determined by filling the lagoon with water to a depth of 15 feet. The decrease in liquid head and the evaporation rate were then to be used to determine the as-built liner permeability. Unfortunately, the as-built liner permeability was much greater than expected. Actual as-built permeabilities were never determined, however, because the water flowed through the liner faster than it could be pumped into the lagoon. Examinations of the liner profile revealed large discontinuities in the liner materials (see Figure 7-25), which explains the high permeability.

A second attempt to construct the liner to the same specifications was successful. All phases of liner installation (by a different contractor) were carefully monitored by the site owner. Final permeability testing confirmed that the liner exceeded the required specifications. A cross-section of this liner revealed uniform thickness and material content (see Figure 7-26).

A complete discussion of CQA/CQC may be found in Chapter 5 of this document.

7.6 WASTE TYPES

Prior to recent Federal regulations (Federal Register, 1982), unsolidified liquid wastes either containerized or in bulk form were often disposed of in landfills. Since these recent regulations, free-standing liquid wastes are required to be removed from drums and solidified prior to final disposal. While the option still exists to store liquids in surface impoundments or evaporation ponds, free liquids must be removed prior to final facility closure.

7.6.1 Free Liquids

Many performance problems have resulted from the practice of placing containerized or bulk free liquids in waste disposal facilities. With bulk liquids, a liquid head is imposed on the liner. If the head becomes too great or if the waste material is incompatible with the liner material, the waste has a much greater chance of infiltrating into or through the liner.

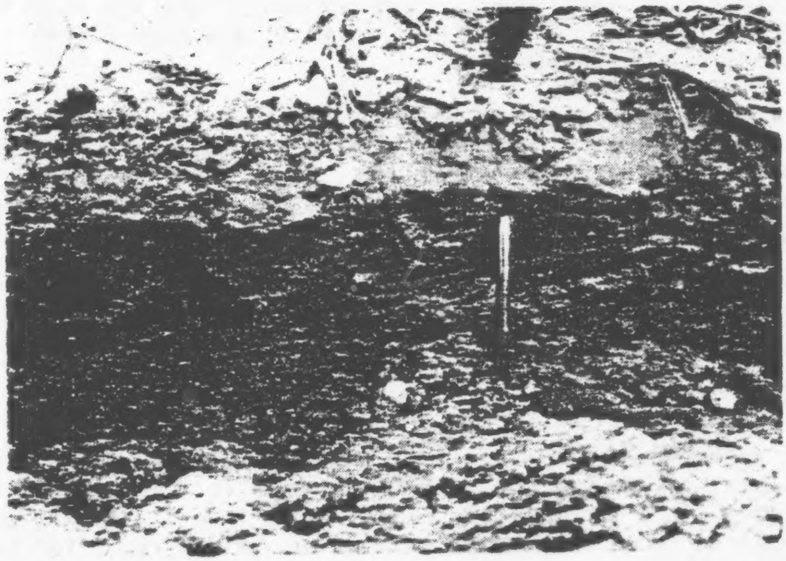

Figure 7-25. Bentonite/soil liner constructed without adequate CQA.

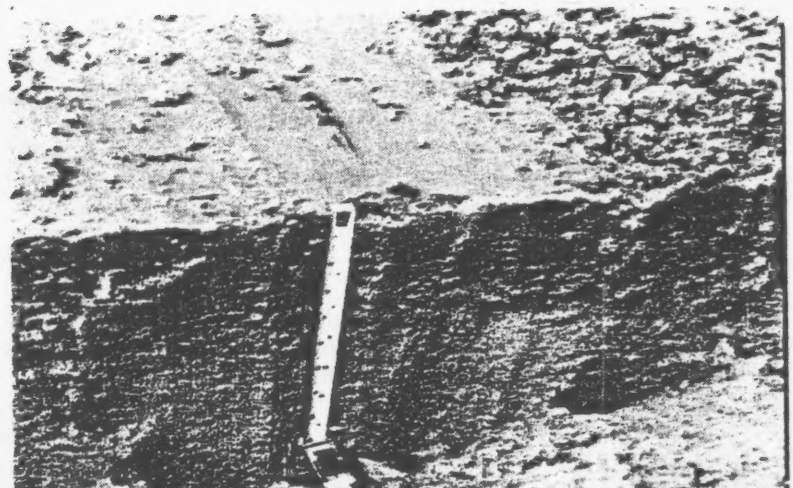

Figure 7-26. Bentonite/soil liner installed with extensive CQA.

Fifty-five-gallon drums, which are often used to contain free liquids, have the potential to degrade with time. This can result in leaky drums, which, in turn, may cause leachate levels to build and the performance problems associated with this phenomenon to occur. Several problems are also associated with the waste placement and closure operations of a facility that contains drums. Drums are usually stacked upright in several layers or lifts. Absorptive material such as soil is then placed between the drums and compacted to fill all voids. It is difficult to determine when all of the lower voids are filled and when the material has been compacted properly. If the spaces are not completely filled, piping into the spaces between the drums may occur. This can result in differential settlement and cap failure, which may lead to increased liquid infiltration and hydraulic gradient on the liner, thus affecting its performance.

An example of a facility where a performance problem has resulted, at least in part, from free liquid disposal is site B. This facility is approximately 13 years old. It is a zone-of-saturation (intergradient) facility where liquids were not solidified prior to disposal. These unsolidified liquids along with the accumulated precipitation and seepage resulted in a "bathtub" with leachate seeps penetrating the cap/liner interface.

Present landfill regulations require that leachate collection and removal systems must be installed immediately above the liner. These systems must be designed to ensure that the leachate depth does not exceed 30 cm. Properly functioning leachate collection systems along with the regulation preventing free liquids from being disposed of in landfills should prevent similar future performance problems from occurring.

7.6.2 Stabilized or Solidified Liquids

As previously mentioned, Federal regulations require that free liquids not be placed in landfills. In response, free liquids are now removed, solidified, or stabilized prior to their final disposal. Removed liquids may be recycled, impounded, used as fuel, or in some way treated and then placed in a landfill either in drums or bulk. The solidification process may be accomplished by mixing the liquid with absorptive material such as lime, cement kiln dust, fly ash, or some other substance. Stabilization of wastes may include processes such as neutralizing acidic or basic wastes or precipitating heavy metals. These processes generally decrease the toxic or hazardous nature of the waste and are currently practiced at many of the case study sites in Section 7.2.

A procedure that provides protection from leaking drums is practiced at site E. Here, a "calculated amount" of "absorptive material" is placed beneath each lift of waste. This material will theoretically absorb any liquids that might leak from the drums. The success of this method is not proven at this time as site E has had large volumes of leachate accumulating in some of its waste trenches.

7.6.3 Sludges and Solid Wastes

7.6.3.1 Sludges--

Depending on the water content, sludges may be either landfilled or placed in evaporation ponds. However, in both cases, it is important that most of the moisture be removed prior to final facility closure. Case studies M, P, and Q are all landfills that contain various types of sludges. All of these facilities contain both leachate collection systems and some form of leak detection system.

Site M uses a special procedure for waste placement. This procedure involves mixing a thin layer (approximately 2 feet) of locally available soil with a thin layer of the inorganic lime sludge, thereby reducing the overall moisture content of the material and giving it structural stability. The range of the soil-to-waste ratio varies from 0.5:1 to 1.5:1. Exact mixing proportions are determined in the field and judged sufficient when the landfill equipment is able to drive over the mixture.

Facilities P and Q contain brine purification mud and metal hydroxide and waste pigment sludge, respectively. These facilities are both lined with bentonite/soil admixtures. The bottom and lower side liners at both of these facilities are thicker than the upper side liners to provide extra protection in areas where a liquid head may be present.

The following case study is not included in Section 7.2 due to insufficient information. Enough information is available, however, to allow a failure that occurred at this facility to be attributed to improper waste placement procedures. This double-lined facility was originally designed for the disposal of solid wastes. A change in the parent company's waste disposal needs resulted in this facility accepting high-liquid-content sludges. The sludges were transported to the site in tank trucks that would back up to the facility and dump their load through a pipe whose outlet was well above the base of the liner. The impact of the sludge on the liner caused severe liner erosion and groundwater contamination probably as a result of an erosional breach in the liner.

7.6.3.2 Solid Waste--

Solid wastes may be landfilled in containers such as 55-gallon drums or in bulk form. When wastes are landfilled in drums, all free liquids must be removed or solidified prior to final disposal. Care must be exercised to make certain that the voids between adjacent drums are filled with well-compacted soil or other material. Problems that may result from the disposal of drums are discussed in Section 7.6.1.

Solid wastes such as contaminated soil or other fine uniform material may be emplaced and compacted in lifts. When this method is used, the void volume in the landfill is reduced; thus, future problems due to settlement or piping are less likely to occur. Solid wastes that are not uniform, such as unshredded tires, scrap metal, and municipal solid wastes, on the other hand, are more difficult to compact thoroughly and therefore have the potential to settle differentially. This can be minimized by crushing or shredding the waste, thereby reducing its void spaces and overall volume. The use of a landfill compactor will also help to prevent future problems due to piping and settling.

7.6.4 Waste Compatibility

7.6.4.1 Waste/Liner Compatibility--

Laboratory tests suggest that dilute aqueous leachates will not affect the permeability of clay liner materials if the moisture at compaction is uniform and close to optimum and if compaction is uniform. However, data from several studies suggest that drastic increases in permeability can result from certain clay-chemical interactions. Examples of organic chemicals that have been demonstrated to increase clay soil permeability are: aliphatic and aromatic hydrocarbons (e.g., cyclohexane, heptane, kerosene, naphtha, benzene, and xylene), alcohols (e.g., methanol and ethylene glycol), ketones (e.g., acetone and dioxane), amines (e.g., aniline and pyridine), carbon tetrachloride, and nitrobenzene. Changes in permeability have also been noted with strongly acidic permeant fluids.

Bentonite is sometimes given a polymer treatment to improve its resistance to the effects of normally incompatible fluids. The long-term viability of polymer-treated products needs to be verified.

The enormous variability in clay soil from different locations complicates the task of predicting clay-chemical compatibility. Data from several laboratory studies suggest the possibility of drastic increases in permeability as a result of certain clay-chemical interactions. It should be emphasized, however, that many aqueous leachates have been tested and produced no significant increases in permeability. For a detailed discussion of clay-chemical interactions, see Chapter 4 of this document.

Facilities C and J have both had liner failures that have been attributed to liner/waste incompatibility. In the case of site C, an investigation of groundwater contamination identified the facility's treatment ponds as the probable source of the pollution. This conclusion was based on the fact that the materials in the treatment ponds were usually of low pH and would have reacted with carbonate seams and inclusions in the surrounding soil. Several of the disposal trenches at this facility were also used for the disposal of highly acidic waste, in this case oil reprocessing sludges. These acidic sludges may have also reacted with the carbonate inclusions, creating paths for leachate seepage and groundwater contamination.

The liner failure at site J occurred almost overnight. Samples of liquid removed from the leak detection system at one of the ponds on one day were reported to be "clean." Samples taken the following day contained over 11.2 percent perchloroethylene plus small amounts of other chlorinated and nonchlorinated organics. The facility was allegedly not accepting wastes with more than a trace of perchloroethylene.

This subcell has a pH buffer system that maintains pH levels between 6 and 8.

- Heavy metals. These wastes represent approximately 15 percent of the total waste volume accepted at site F. This group is comprised of all heavy metals and asbestos. This subcell contains the smallest amount of organic materials, which helps to reduce fire hazards caused by the reaction of strong oxidizing agents with organics.

- Highly flammable wastes. This type of material represents approximately 12 percent of the total waste volume accepted at site F. These materials generally exhibit a flash point between 80 and 100 °F. These materials are kept apart from powerful oxidizing agents, materials that are prone to spontaneous heating, or materials that react with air or moisture to evolve heat.

- Toxic materials. These wastes represent approximately 23 percent of the total waste volume accepted at site F. Included in this category are all waste materials containing more than 5 percent by weight of highly toxic organic compounds, carcinogens, PCB's, and other halogenated wastes. No solvent-type wastes are permitted in this subcell.

7.7 PERFORMANCE MONITORING

Several methods can be used to monitor the performance of clay-lined landfills. These methods generally involve groundwater monitoring or monitoring of the unsaturated zone directly beneath the clay liner. Monitoring the quantity and quality of the leachate above the liner may also give some indication of the facility performance. This type of monitoring program for a facility should be able to detect performance problems, such as leachate migration, soon after they occur. This will enable the rapid remediation of any problems and prevent serious groundwater contamination.

7.7.1 Unsaturated Zone Monitoring

Two systems that are commonly used to monitor the unsaturated zone beneath a clay liner are lysimeters and continuous-coverage leak detection systems. A continuous-coverage leak detection system generally consists of a flexible membrane overlain by a porous layer that contains perforated collection pipes. Just as with a leachate collection system, the porous layer is usually sloped so that any detected fluid will drain to a sump. Here the liquid may be sampled and analyzed for hazardous constituents. Total liquid volumes removed from a detection layer may also be used to estimate the liner permeability.

An investigation into the problem revealed that aqueous wastes coming from one of the disposal facility customers contained approximately 5 percent degreasing fluid waste that contained perchloroethylene. This waste was to have been stored in a separate tank at the originator's plant. Evidently, it was not made clear to the workers where to put the "perc" waste, and it was placed in an aqueous waste storage tank. Being insoluble and heavier than water, it sank to the bottom. When a tank truck from the disposal facility would collect a load of aqueous waste, it would also receive a few hundred gallons of the "perc" waste. The waste sampling method used by the disposal facility on all loads of incoming waste was not able to sample the bottom few inches of each load. In addition, the parking area where all incoming truckloads of waste were sampled was slightly sloped, causing the relatively small amount of "perc" waste in the bottom of the tanker to flow to the back of the truck where it could not be reached by the sampling device.

Over a period of approximately 9 months, the aqueous waste containing "perc" was placed into one of the clay-lined ponds and went unnoticed until the liner failure occurred. The apparent incompatibility is consistent with current research on the effects of chlorinated solvents on clays.

7.6.4.2 Waste/Waste Compatibility--

Large disposal facilities that accept most types of wastes, such as sites E and F, generally have a waste placement plan. Such a plan is used to separate incompatible or reactive materials and in some cases to place together materials that produce "favorable reactions."

Most cells at site F are divided into five subcells. The purpose of these subcells is to isolate the various waste groups accepted at the facility, thereby preventing the interactions of incompatible wastes. The waste categories, percentage of total wastes, and separation rationale at site F are as follows:

- General wastes. These wastes represent approximately 44 percent of the total waste volume accepted at site F. General wastes are defined as materials of both an organic and inorganic nature that do not contain a significant quantity of any of the other waste categories.

  The hazardous acidic or acid-generating materials are covered with lime to ensure that any acid that is generated will be neutralized.

- Pseudo metals. This type of material represents approximately 6 percent of the total waste volume accepted at site F. Pseudo metals are arsenic, antimony, bismuth, and phosphorous. Chalcogens, beryllium, and any of their compounds as well as alkaline-sensitive materials are also disposed of in this subcell.

The advantages of a continuous-coverage leak detection system include its ability to provide early indications of the liner performance and its ability to detect and collect all liquids that appear in the detection system. The disadvantages of a continuous-coverage detection system include its relatively high cost and the possibility that the leachate, if a chemical compatibility failure should occur, might spread out upon and damage a large portion of the lower clay liner or foundation soil.

Another approach to unsaturated zone monitoring involves the use of either pressure-vacuum lysimeters or collection lysimeters. Pressure-vacuum lysimeters consist of a porous cup, placed in a bore hole, that a vacuum may be applied to. This vacuum causes pore liquids to collect in the porous cup. The liquid is then removed and analyzed for hazardous constituents. A collection lysimeter, on the other hand, is a lined gravel or sand-filled basin or trench beneath the primary liner. A collection pipe located at the low point of the lysimeter leads to a sump from which any detected liquids can be sampled and analyzed.

The following discussion on the advantages and disadvantages of collection and pressure-vacuum lysimeters is taken from Kmet and Lindorff (1983).

> Both pressure-vacuum lysimeters and collection lysimeters can be used to monitor the unsaturated zone beneath a landfill and provide an early indication of landfill performance. Each has its own advantages and disadvantages.
>
> The major advantage of a collection lysimeter is that it provides a method of monitoring the quantity as well as the quality of leakage from the base of a landfill. If one assumes that the leakage reaching the lysimeter is representative of leakage over the entire base of the site, it is possible to calculate the volume of leakage from the entire landfill and thus assess the effectiveness of the liner in attenuating the leachate and in limiting the rate of leachate movement.
>
> Also, collection lysimeters do not require the special equipment or the care necessary to sample pressure-vacuum lysimeters properly. Samples can be taken from a manhole or riser by a bailer rather than a vacuum pump. In addition, collection lysimeters do not require that a vacuum be placed on the system at least several days prior to sampling, as is the case with pressure-vacuum lysimeters. When the monitoring results from a landfill indicate that one or more pressure-vacuum lysimeters are dry, one is never sure whether there was insufficient soil moisture or whether improper sampling techniques were used. With collection lysimeters, one is reasonably certain that if liquid is present in the subsurface, it will be collected. Collection lysimeters also eliminate the problem of frozen lines in cold weather that can occur with pressure-vacuum lysimeters.

Research, including work done by Apgar and Langmuir (1971); Johnson and Cartwright (1980), has suggested that some chemical constituents may be filtered out by the porous membrane of the pressure-vacuum lysimeter, thereby producing inaccurate water quality data. This is not a concern with collection lysimeters since no membrane is present.

The major disadvantage of a collection lysimeter in comparison with a pressure-vacuum lysimeter is the lower cost and shorter time required for pressure-vacuum lysimeter installation. Whereas a pressure-vacuum lysimeter is installed in a bored hole, construction of a collection lysimeter necessitates excavation of sufficient area to appropriate grade to install the basin, collection pipe, and manhole. A substantial number of pressure-vacuum lysimeters could be installed in the time and for the cost of one collection lysimeter. A pressure-vacuum lysimeter can be installed for less than $100 plus drilling costs. Although data is [sic] limited, the cost of collection lysimeter installation, including excavation, materials, and construction costs, is expected to be several thousand dollars and can approach $10,000 for the larger lysimeters.

Another disadvantage is that only one collection lysimeter can be installed at a given location. Therefore, data is [sic] generated for only one depth in the unsaturated zone, typically just below the clay liner. In contrast, pressure-vacuum lysimeters are frequently installed in a nest at different depths to monitor changes in water quality with depth.

Several of the facilities discussed in Section 7.2 of this document have some type of unsaturated zone monitoring system. Six of these facilities--sites H, L, M, N, P, and Q--are landfills and three--sites I, J, and K--are ponds. Sites H and N both have lysimeters beneath their clay liners. Facility H has two 1,200-ft^2 lysimeters directly beneath the 4-foot-thick recompacted clay liner. An additional lysimeter is located under the leachate storage basin. For approximately 9 months after the start of landfilling operations, small quantities of liquid were removed from the lysimeters under the landfill liner. These liquid volumes declined steadily over time and were reduced to zero 1 year after the start of landfill operations. The liquid that was initially collected in the lysimeters is most likely soil moisture that was released after liner construction.

Facility N also has two lysimeters beneath the liner. This facility also has a groundwater drain that is used to lower the groundwater table. An interesting fact about this facility is that although highly contaminated liquids have recently been removed from the lysimeters, contaminated liquids were first discovered in the groundwater drain manhole. This has continued to be the location where the majority of contamination has been detected.

The three ponds with unsaturated zone monitoring all have continuous-coverage leak detection systems between two recompacted clay liners. At two of the ponds, facilities I and J, major performance problems have developed in the upper clay liners. In both cases, the problems were quickly detected (overnight at site J) and corrected, thus preventing any contamination from penetrating the lower liner. The third double-lined pond, site K, has had small quantities of liquid accumulating in its leak detection system. These volumes were used to calculate the overall liner permeability, which ranged from 4×10^{-8} cm/s to 3×10^{-7} cm/s, demonstrating that the original specifications of 1×10^{-6} cm/s had been met.

Four of the six landfills, facilities L, M, P, and Q, have continuous-coverage leak detection systems. Liquids have been detected in the leak detection layers of sites L, P, and Q. However, only one, site L, is considered a failure. Here, slightly contaminated liquids were removed from the detection system manhole less than 1 year after the start of operation. Facility Q also had slightly contaminated liquids removed from its detection system. The volume of liquid removed was used by the facility owner to calculate the admixed liner's permeability, which ranged from 3×10^{-8} cm/s to 6.5×10^{-8} cm/s, indicating that the liner met the original permeability specification of 1×10^{-7} cm/s. The as-built liner permeability at facility P has not been determined. The source of small amounts of uncontaminated liquids in the detection system at this facility was being investigated by the site owner. The results of this investigation were not available at this writing.

7.7.2 Groundwater Monitoring

Information on Federal groundwater monitoring requirements may be found in "Groundwater Monitoring Guidance for Owners and Operators of Interim Status Facilities" (U.S. EPA, 1983).

Groundwater monitoring wells are the most commonly used type of performance monitoring systems. They are relatively inexpensive to install and maintain, and samples are easily obtained from them. They may be placed in areas where contamination is estimated to be most likely to occur. They may also be placed in groups or clusters with each well being at a different depth and therefore monitoring the vertical as well as lateral movement of any contamination. Generally, wells are placed both upgradient and downgradient from a facility prior to the start of operations. This enables baseline data for both upgradient and downgradient wells to be obtained. Future groundwater samples may then be compared with the baseline data and facility performance can be estimated.

Two major problems with relying solely on monitoring wells to determine facility performance are that leachate plumes may be missed and that the monitoring data are only qualitative. If an adequate number of wells at varying depths are used, the possibility of missing a contaminant plume is greatly decreased.

An additional problem that may occur with monitoring wells is poor construction procedures such as the use of inappropriate materials. This can often lead to well-water contamination that is incorrectly attributed to the waste disposal facility. Examples of such problems occurred at sites A and E. Groundwater samples taken from the wells at site A in 1976 had pH levels between 10.7 and 11.3, COD's of more than 3,000 ppm, and chloride levels greater than 1,000 ppm. Originally, it was thought that this contamination originated in the disposal facility. A complete analysis of the problem, however, revealed that the monitoring wells were improperly installed and contained steel-mill slag where clean gravel should have been used. Analysis of samples taken from new wells, constructed according to the specifications, has not indicated elevated levels of any hazardous constituents.

Monitoring well analysis at site E has shown trace levels (92 ppb maximum) of 12 organic compounds in the groundwater. An investigation of their source indicated that the organics most probably came from the plastic well casing materials. This case study points out the importance of testing the compatibility of all materials with the local groundwater as well as the expected leachate.

7.7.3 Leachate Level and Quality Monitoring

Leachate collection systems are designed to remove leachate that would otherwise accumulate in the liner. By doing this, they prevent the buildup of leachate head, thus keeping liquid infiltration into the liner to a minimum and therefore contributing to the overall performance of the liner.

Some facilities incorporate leachate level and leachate quality monitoring into their overall monitoring program. These parameters serve as indices of the efficiency of treatment methods that they may have been used on waste materials and the leachate collection system. Analysis of the leachate may be used to predict the quality of the liquid, if any, that may be passing through the liner. Leachate and liner samples can be subjected to laboratory compatibility tests to indicate the facility performance.

7.8 CONCLUSIONS

The sites included in our study and a brief description of their clay liners and performance are presented in Table 7-1 and are fully described in Section 7.2. The 17 sites included in this section were chosen because they illustrate a specific performance problem or facility characteristic. While conducting our information search, we encountered many facilities both with and without failures that were not included here because they would not have provided any additional information.

Comparing the data from successful sites with the data from leaking sites does not reveal any significant differences in clay or liner characteristics that might explain the differences in performance. Similarly, performance does not seem to be related to the use of recompacted borrow clay versus recompacted native clay or admixed materials. The performance of facilities without recompacted liners is, however, generally poorer than the performance of facilities with a recompacted clay or admixed liner.

Also note that many of the sites discussed in this section are relatively new (after 1975). It is therefore difficult to state whether the facilities without detection systems are performing well or poorly as sufficient time for leaching chemicals to reach monitoring wells may not have elapsed. Performance of the facilities that have detection systems, on the other hand, is much more quickly determined than is the case with facilities I, J, L, N, and Q. Sites I, J, L, and N have reported performance problems. In all cases, the leak detection system was instrumental in the rapid detection of the performance problems. Site Q has had small amounts of liquid detected in its leak detection system. The volume of liquid removed was used to determine the permeability of site Q's upper liner. This value was lower than the design specification, indicating good performance. Of the 17 facilities discussed in Section 7.2, 6 have had major liner failures. Of these six, the cause of four of the failures is known and the cause of the failure of the remaining two is unknown. Generally, the cause of a failure in a clay-lined landfill is very difficult to determine because this would require removing the waste and other overburden, which may in itself cause liner damage and impose further hazards. Determining the cause of a liner failure in a lagoon or evaporation pond is somewhat less difficult because the waste is more easily removed.

The four facilities with explained failures include three lagoons or ponds and a landfill that accepted unsolidified liquids. Two of these facilities had leak detection systems; the others did not. The failures at the facilities with detection systems were noticed soon after they occurred, thus preventing groundwater contamination. The other two failures were not detected until groundwater contamination was found in a monitoring well. Two of the failures were due to chemical compatibility problems, one to leachate migration through an improperly sealed sand seam, and the other to desiccation cracks that formed in an unprotected liner.

Sites with a single layer of clay or those relying on in situ clay formations are less secure since a leak cannot be detected until pollutants are found in groundwater samples. The security of a landfill would be greatly increased if it were constructed with a continuous-coverage leak detection system installed between two layers of clay. If the inner layer failed, the leak detection layer would serve as a buffer zone that might preserve the integrity of the outer layer. The use of lysimeters as a method for determining liner performance also may provide an early indication of leachate migration. However, these devices are not foolproof because they are only able to detect leaks in the section of the liner directly above them.

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CHAPTER 8

CLAY LINER TRANSIT TIME PREDICTION METHODS

8.1 INTRODUCTION

Although recompacted clays may have low hydraulic conductivity, they are, nonetheless, permeable porous media. If a finite depth of liquid waste or leachate is maintained indefinitely over a clay liner, the liquid and leachate chemicals will eventually seep through. The goal of transit time prediction methods is to determine both the rate of seepage with time and the time it will take for liquids to seep through a liner.

Generally, the transit time prediction methods discussed in this chapter may be used in two ways. First, these methods may be used to facilitate the design of new clay liner systems. In fact, the U.S. Environmental Protection Agency (U.S. EPA) has considered using a transit time equation to provide a method for estimating necessary bottom liner thickness as a function of the design facility life. Second, these methods may be used to predict the performance of existing clay liner systems and to determine the potential for groundwater contamination from leachate or liquid wastes.

This chapter examines seven transit time prediction methods and compares their strengths, weaknesses, and applicability to various clay liner conditions. Section 8.2 reviews the assumptions and basic equations that underlie the use of all these methods. Section 8.3, which comprises the bulk of the chapter, discusses the derivation and use of each method. Section 8.4 compares the consequences--i.e., the predicted transit time, in years--of using each method. The methods are compared under two different performance criteria and assuming three different initial moisture contents of the clay liner. The prediction of transit times generated by various models cannot be compared with any actual liner data because no data regarding liner infiltration and breakthrough under certain conditions are currently available. Section 8.5 offers a summary and some conclusions regarding the use of the different methods.

Many of the terms used in this chapter have specific technical meanings, and Section 8.6 provides a list of these terms along with their definitions. References are contained in Section 8.7.

8.2 BACKGROUND CONSIDERATIONS

8.2.1 Performance Criteria

It is necessary to establish specific performance criteria that will provide a definition for "transit time" to design or evaluate a liner system. These performance criteria should be designed so that if they are satisfied, the liner system will ensure safe operation over the intended life of the facility or up to a certain time after closure. Performance criteria may be expressed in terms of seepage flux, contaminant flux (indicated by the concentration of a chemical that is present in the leachate), or the time taken to reach a specified flow rate or chemical concentration at the bottom of the liner. The following are five examples of possible specific liner performance criteria:

- Specified seepage flux at the liner bottom
- Specified contaminant flux for a particular leachate component at the liner bottom
- Time taken to reach a specified seepage flux at the liner bottom
- Time taken to reach a specified leachate chemical flux at the liner bottom
- Time taken for the concentration of a specific leachate component to reach a specified value at the liner bottom.

Selection of specific performance criteria is very important in predicting corresponding "transit times," and it can also influence the applicability of a given method. A wide variation in the predicted transit times was observed in the present work (as shown in Table 8.2 in Section 8.4.1) based on performance criteria selected and models used.

Recommendations regarding the use of specific performance criteria are beyond the scope of this work. However, examples of the consequences of different performance criteria will be considered in the evaluation of transit time prediction methods.

8.2.2 Clay Liner System

The equations and boundary conditions used in transit time prediction methods are based on a specific site geometry or flow domain. This flow domain describes the path of waste leachate through a clay liner system. For the sake of this discussion it consists of, in a vertically downward direction, saturated solid waste; a sand/gravel bed on top of the clay liner (leachate collection system); a clay liner; a sand/gravel leak detection system; underlying local soil; and an underlying saturated aquifer zone. Current regulations (40 CFR 264.301(a)(2)) require a leachate collection system on top of a clay liner to maintain a leachate head of less than 1 foot. This makes it unnecessary to include the saturated solid waste zone and the leachate collection system zone in the flow domain. The sand/gravel leak detection layer may not be present in some designs. A schematic of the flow domain that is used to illustrate the transit time equations discussed below is shown in Figure 8-1.

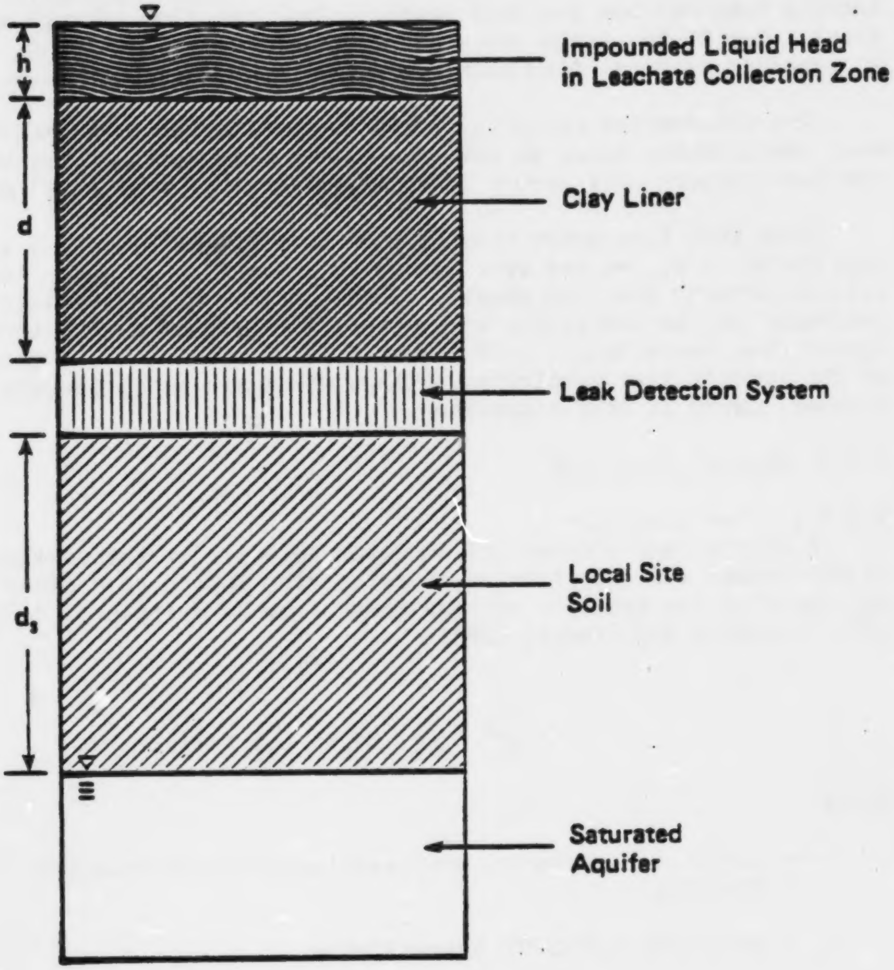


Figure 8-1. Flow domain for leachate flow.

Because the hydraulic conductivity of the clay liner material is much lower than the underlying site soil, the rate of leachate flow is predominantly controlled by the clay liner with the underlying soil influencing only the boundary conditions at the bottom of the clay layer. Some of the transit time methods are thus based on leachate flow through the clay liner alone. Due to the large ratio of liner width to liner depth, it is adequate to consider vertical flow alone to describe the leachate flow.

Any attenuation capacity of the clay or temporary immobilization of some contaminants tends to slow the leachate migration. Failure to account for such factors will result in underestimates of transit time.

Note that throughout this section only aqueous leachate systems are considered; i.e., we are considering only the flow of water and dissolved species through the flow domain. Although some organic-solvent-based leachates may be immiscible with water, analysis of such a three-phase system (two immiscible liquid phases + air) is extremely complex and none of the transit time prediction methods discussed below (except numerical methods) apply to this situation.

8.2.3 General Equations

8.2.3.1 Flow Equation--

A general one-dimensional equation to describe the vertical flow of fluid through the saturated/unsaturated flow domain at a given position z, but ignoring the specific storage terms, may be written as follows (Bear, 1979; Huyakorn and Pinder, 1983):

$$\phi \frac{\partial S_w}{\partial t} = \frac{\partial}{\partial z} \left[K_s k_r \left(\frac{\partial \Psi}{\partial z} - 1\right)\right] \tag{8.1}$$

where

z = vertical coordinate, expressed as positive downward distance

K_s = saturated hydraulic conductivity

k_r = relative hydraulic conductivity with respect to K_s

Ψ = pressure head

ϕ = porosity

S_w = fractional saturation and is equal to 1 for saturated media

t = time.

The moisture content, θ, is given by (ϕS_w). The saturated hydraulic conductivity depends upon the porous media (clay, soil, etc.) and the composition of the leachate solution. (Clays may have substantially higher permeabilities to some organic solvents or inorganic chemicals than to water. See Chapter 4.) The relative conductivity, k_r, is predominantly a function of moisture content and depends upon soil type. The pressure head, Ψ, is also predominantly a function of moisture content and soil type; in addition, it includes the effect of the liquid head on top of the liner. Porosity, ϕ, depends upon the soil type and is related to the pores open to the fluid flow.

Equation (8.1), the generalized flow equation, describes moisture movement in the flow domain. For saturated environments, the equation transforms into a steady state formulation and one can determine the steady state flux. For unsaturated conditions, this equation describes the advancement of the saturated wetting front and also the unsteady state fluid flux as a function of position. Fluid source and sink terms have not been included in the above formulation because they are not likely to be important.

8.2.3.2 Solute Transport Equation--

Equation (8.1) does not consider the transport of soluble chemicals in the leachate. A general equation to describe the vertical transport of a nonradioactive solute species may be written as follows (Bear, 1979; Huyakorn and Pinder, 1983):

$$\frac{\partial}{\partial t}(R\phi S_w C) = \frac{\partial}{\partial z}\left(D\frac{\partial C}{\partial z}\right) - \frac{\partial}{\partial z}(vC) \qquad (8.2)$$

where

D = dispersion coefficient that includes both molecular diffusion and mechanical dispersion

C = solute concentration in liquid phase

z = vertical coordinate, expressed as positive downward distance

v = Darcian velocity in z direction

R = retardation factor to account for attenuation capacity of the clay or soil.

The source and sink terms are not included because they are not expected to be significant. For a conservative species, the retardation factor equals 1. The Darcian velocity, v (cm/s), is obtained from the flow equation and is given by:

$$v = -K_s k_r \left(\frac{\partial \Psi}{\partial z} - 1\right) \quad . \qquad (8.3)$$

8.2.3.3 Parameters Involved in General Equations--

The preceding governing equations involve several physicochemical properties of porous media that must either be determined experimentally or estimated from available data for similar soils. The transit time prediction methods discussed here make certain assumptions to simplify these equations. As a result, a certain method may require only a few of these parameters. Table 8-1 identifies the different methods presented here and the physicochemical properties required for each. It also includes several references that offer additional information regarding each parameter.

The saturated hydraulic conductivity, K_s, and porosity (effective), ϕ, of different layers of homogeneous porous media in the flow domain are required by all methods. Several methods, such as fixed- and flexible-wall permeameters, are available for measuring the saturated hydraulic conductivity of a soil, although no standardized American Society of Testing Materials (ASTM) method exists to date. A review of available methods is given in Section 3.8 of this Technical Resource Document (TRD) and in another draft TRD entitled "Soil Properties, Classifications, and Hydraulic Conductivity Testing" (in preparation). Due to clay-chemical interactions and their effect on clay permeability (see Section 4), it is recommended that hydraulic conductivity be measured with a representative leachate solution.

Methods like the gas expansion method and mercury porosimetry are available to determine total porosity; however, it is the effective porosity that is important for flow dynamics. Methods to determine effective porosity are still at research-tool levels, and are discussed in the above-mentioned draft TRD. Transit time prediction methods dealing with the unsaturated flow dynamics in the clay liner use the term "saturation moisture content" rather than porosity of the medium.

ASTM standard methods are available for measurement of moisture suction potential, $\Psi(\theta)$, of soil media. Method No. D-2325 refers to coarse- and medium-grained soil materials, whereas No. D-3152 refers to fine-grained materials such as clay (ASTM, 1985).

Like the suction potential, $\Psi(\theta)$, the relative hydraulic conductivity, K_r, with respect to saturated hydraulic conductivity, is also a function of moisture content and soil type. Measurement of relative hydraulic conductivity is more difficult and involved than that of saturated hydraulic conductivity and, to date, there are no standardized ASTM methods to measure K_r. A review of currently available techniques is presented in "Soil Properties, Classifications, and Hydraulic Conductivity Testing."

The solute transport equation involves two additional parameters: retardation factor, R, and the axial dispersion coefficient, D. Retardation factor is dependent upon the attenuation capacity (e.g., adsorption) of the

TABLE 8-1. PARAMETERS REQUIRED IN DIFFERENT TRANSIT TIME PREDICTION METHODS

Transit time prediction method	Parameter					
	Saturated hydraulic conductivity, K_s	Effective porosity, ϕ	Moisture suction potential, ψ (0)	Relative hydraulic conductivity, K_r $(0)_1$	Dispersion coefficient, D	Retardation factor, R
Simple transit time equation	✓	✓				
Modified transit time equation	✓	✓				
Green-Ampt wetting front model	✓	✓	✓			
Transient linear-ized infiltration model	✓	✓	✓	✓		
Modified transit time equation with diffusion	✓	✓			✓	✓
Numerical methods:						
Flow equation	✓	✓	✓	✓		
Solute transport equation	✓	✓	✓	✓	✓	✓
References	Section 3.8 a,b,c,d	a,b,c,d	ASTM method[e] No. D-2325 D-3152 Vol. 04.08	a,b,c,d	b,c,d	Section 4 b,c,d

[a] U.S. EPA (1985, draft).

[b] Bear (1972).

[c] Greenkorn (1983).

[d] Freeze and Cherry (1979).

[e] ASTM (1985).

soil medium and needs to be determined experimentally. Note also that the attenuation capacity of soil is time dependent and it is reduced as the soil approaches saturation with respect to adsorbed species. For conservative liner thickness predictions, it is probably best to assume retardation factor of unity (no attenuation).

The dispersion coefficient consists of two parts: hydrodynamic dispersion and molecular diffusion. The hydrodynamic dispersion coefficient is dependent upon flow velocity, whereas the molecular diffusion coefficient is not. The molecular diffusion coefficient should be readily available from the literature, whereas measurement of the hydrodynamic dispersion coefficient is tedious and time consuming, and no standardized methods are available for this purpose.

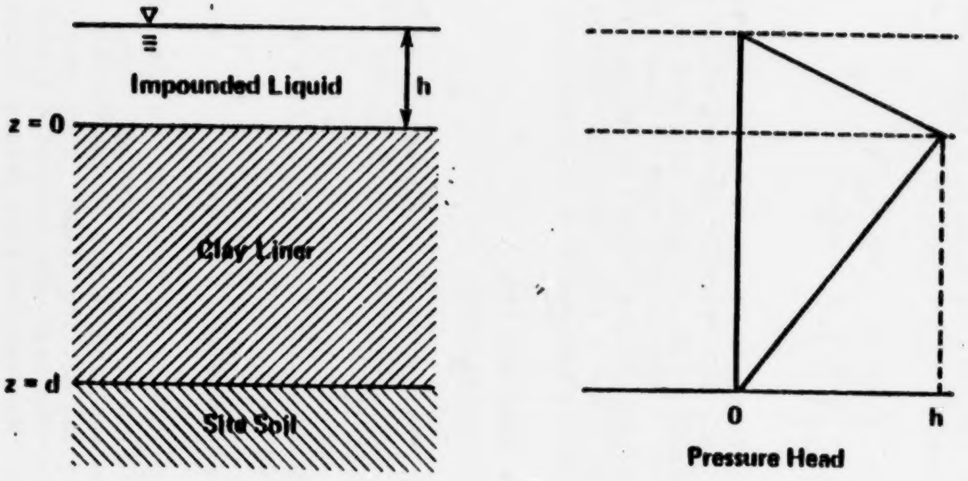
8.3 TRANSIT TIME PREDICTION METHODS

Equations (8.1) and (8.2) describe leachate flow and solute transport in the flow domain shown in Figure 8-1. Solving these equations will give the seepage flux, leachate flux, moisture profiles, and concentration profiles as a function of time. Together with the appropriate performance criteria, these equations may be used to assess the performance of an existing facility or to design an effective new facility. These equations are complex, and in this form they can only be solved numerically; but with certain assumptions it is possible to simplify the equations and allow analytic solutions. Each of the different transit time prediction methods discussed below uses some form of simplification to achieve this end. In a report attached as Appendix A to U.S. EPA (1984), Cogley et al. reviewed several transit time equations and developed the groundwork for the discussion that follows.

8.3.1 Simple Transit Time Equation

A simple transit time equation is under consideration by EPA as a method to estimate the necessary bottom liner thickness as a function of the design impoundment life (U.S. EPA, 1984, Appendix A). This equation assumes a flow domain that includes only a saturated clay liner. Figure 8-2a shows a schematic of the modified flow domain, and Figure 8-2b graphically shows the assumed pressure potential profile. The vertical axis in Figure 8-2b corresponds to a location in the liner flow domain shown on the left. Due to saturated conditions, Equation (8.1) is transformed into a steady state equation, and the steady state Darcian velocity (cm/s) or flux (cm^3/cm^2s) is given by Equation (8.3). At the top of the liner ($z = 0$), the liquid pressure head is equal to the impoundment liquid head ($\Psi = h$); at the liner bottom, z is equal to liner thickness ($z = d$). Because the liner is saturated, the pressure head is taken as zero ($\Psi = 0$) at the liner bottom, assuming free drainage. Thus, the steady state Darcian velocity and flux, assuming linear potential gradient, are given by:

$$v = K_s \left(\frac{h}{d} + 1\right) . \qquad (8.4)$$



a) Schematic of Liner

b) Steady State Pressure Head Profile

Figure 8-2. Schematic of flow domain and assumed steady state pressure head profile: Simple Transit Time Equation.

Because the liner is assumed to be saturated, the seepage flux is established as soon as the impoundment liquid head is established. The time taken by the leachate chemicals at the top of the liner to arrive at the liner bottom under steady state saturated conditions, due to advection only, may be obtained by solving Equation (8.2). Assuming D = 0 and R = 1, Equation (8.2) gives the required transit time, t, as:

$$t = \frac{\phi d}{v} = \frac{\phi d}{K_s}\,\frac{d}{(h+d)} \; . \qquad (8.5)$$

The liner thickness that is required to achieve a given transit time may then be obtained based on Equations (8.4) and (8.5):

$$d = 0.5\left\{\frac{K_s t}{\phi} + \left[\left(\frac{K_s t}{\phi}\right)^2 + \frac{4K_s th}{\phi}\right]^{\frac{1}{2}}\right\} . \qquad (8.6)$$

Because only the advective transport of solute is considered when the solute transport equation is solved, the leachate concentration at the bottom of the liner after the transit time would be the same as that of waste leachate on top of the liner. Before this transit time, the leachate chemical concentration in the seepage at the liner bottom would be zero. The leachate chemical flux at the liner bottom after the transit time can be obtained simply by multiplying the seepage flux (cm^3/cm^2s) by the leachate concentration (ppm).

The key assumptions in this approach are:

- Steady state saturated Darcian flow
- Pore fluid pressure at the bottom of the liner is equal to atmospheric pressure; i.e., the bottom layer of the liner remains saturated indefinitely
- Solute transport is by advection only; i.e., there is no diffusion or dispersion
- Attenuation capacity of the clay is ignored.

Most clay liners are only partially saturated at the time of placement and become saturated due to the flow of leachate through them. Thus, unsteady state, unsaturated flow equations would be appropriate to describe moisture movement during the initial wetting stage. When unsaturated, the clay has lower hydraulic conductivity than when saturated (conductivity is a function of moisture content). However, suction forces induced by capillary tension in unsaturated soils may dominate over gravitational forces. This was pointed out by Moore (1980):

> During early stages of wetting of a compacted clay liner, capillary attraction forces will predominate over gravitational forces. As the clay liner becomes wetter, the capillary forces decrease in importance; and, when the liner is saturated, these forces become negligible in comparison to gravitational forces.

The importance of capillary forces depends upon the initial moisture content of the compacted clay before it is covered by waste material. If the higher hydraulic conductivity under saturated conditions can offset higher potential gradients but lower hydraulic conductivity under unsaturated conditions, the simple transit time equation may give satisfactory results. If, with reduced moisture content, the increase in capillary forces is greater than the corresponding reduction in hydraulic conductivity, this approach will underpredict the required liner thickness.

The simple transit time equation assumes that the pressure head at the liner bottom is zero. However, actual soil underlying the clay liner is likely to influence the conditions at the liner bottom. If a saturated clay liner is placed on an unsaturated soil bed, the soil may exert a suction potential to initiate seepage within the clay liner. Thus, the bottom layers of the liner will not remain saturated, and this suction potential will add to the overall potential gradient and reduce the transit time.

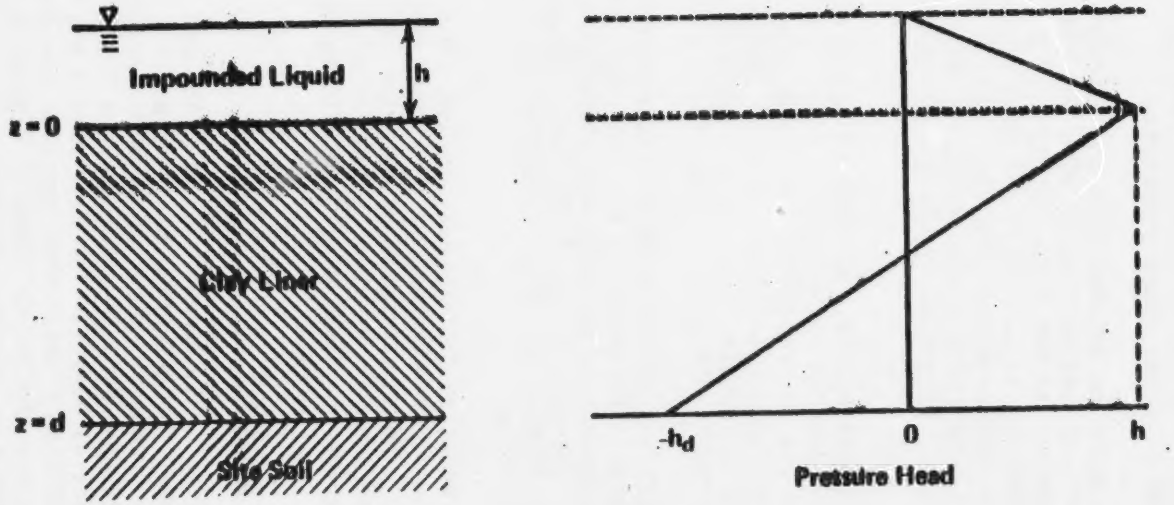
A limiting assumption in this approach is that molecular diffusion or dispersion is not significant compared to convective flux. In view of the low permeability of clays and the corresponding low fluid velocity, the molecular diffusion process may be a significant factor. Diffusion may allow the dissolved chemicals to migrate faster in a liner compared to advective flow and will thus reduce the transit time of the chemical species.

Originally, Equation (8.2) was solved based on total porosity instead of effective porosity (U.S. EPA, 1984, Appendix A). However, effective porosity is more appropriate because it accounts for closed pores and considers only the pores that are open to the fluid flow. This is a relatively simple modification as it does not alter Equations (8.5) and (8.6) but substitutes another value for ϕ.

The simple transit time equation is very easy to use. One needs to know only the saturated hydraulic conductivity of the liner material, its total (or effective) porosity, and the leachate head at the top of the liner. Because these parameters should be readily available, this equation provides a method to obtain quick estimates of liner performance or required liner thickness.

8.3.2 Modified Transit Time Equation

Cogley et al. (in U.S. EPA, 1984, Appendix A) modified the simple transit time equation to account for effective porosity and the suction potential at the liner bottom. However, other assumptions regarding steady state conditions, saturated Darcian flow in a homogeneous liner, and advective solute transport were made as before. The flow domain and the pressure potential profiles for the modified transit time equation are shown in Figures 8-3a and 8-3b. These are similar to those in Figures 8-2a and 8-2b except for the boundary condition at the liner bottom. Due to the steady state saturated conditions, Darcian velocity and flux are again given by



a) Schematic of Liner

b) Steady State Pressure Head Profile

Figure 8-3. Schematic of flow domain and assumed steady state pressure head profile: Modified Transit Time Equation.

Equation (8.3). At the top of the liner ($z = 0$), the pressure head is given by impoundment height ($\Psi = h$), and at the bottom of the liner ($z = d$) the pressure head is negative due to suction ($\Psi = -h_d$). When these boundary conditions are incorporated and a linear potential gradient assumed, the Darcian velocity and flux are given by:

$$v = K_s \left(\frac{h+h_d}{d} + 1\right) . \tag{8.7}$$

Equation (8.2), the solute transport equation, may then be solved as before--assuming saturated steady state conditions and ignoring dispersion and attenuation processes ($D = 0$ and $R = 1$). The resulting transit time for a solute species for a given liner thickness is given by:

$$t = \frac{\phi d}{v} = \frac{\phi d}{K_s} \quad \frac{d}{(h+h_d+d)} . \tag{8.8}$$

The required liner thickness for a specified leachate transit time is then obtained as:

$$d = 0.5 \left\{ \frac{K_s t}{\phi} + \left[\left(\frac{K_s t}{\phi} \right)^2 + \frac{4 \; K_s t \; (h+h_d)}{\phi} \right]^{\frac{1}{2}} \right\} . \tag{8.9}$$

Equations (8.8) and (8.9) are similar to Equations (8.5) and (8.6) except for the h_d term and the use of effective porosity instead of total porosity.

Since only advective transport is considered for solving the solute transport equation, the leachate concentration at the bottom of the liner (after transit time) would be the same as the concentration of waste leachate on top of the liner. Before this transit time, the leachate concentration in the seepage at the liner bottom would be zero. The leachate chemical flux at the liner bottom after the transit time is given simply by multiplying the seepage flux by the leachate concentration.

Like the simple transit time equation, the modified equation assumes saturated steady state conditions and negligible diffusion processes in solute transport. The incorporation of suction potential at the liner bottom is actually somewhat inconsistent with the assumption of saturated flow since the negative pressure would induce unsaturated conditions in lower sections of the liner. Although the suction potential at the bottom is included, the higher hydraulic conductivity value corresponding to saturated conditions is used in this equation. As a result, for a specified transit time, this modified equation yields greater values for liner thickness compared with the simple transit time equation.

In addition to the information required to use the simple transit time equation, the capillary suction at the bottom of the liner ($-h_d$) must be specified. The suction pressure or capillary pressure head depends upon the clay liner material, its particle size distribution, and the underlying site soil dynamics (McWhorter and Nelson, 1979). The underlying site soil, for example, may be assumed to be in stable equilibrium with respect to the water table, in which case the required suction potential will be given by the depth of the site soil above the water table. Depending upon the value of the capillary suction potential, different values of the required liner thickness are obtained.

8.3.3 Green-Ampt Wetting Front Model

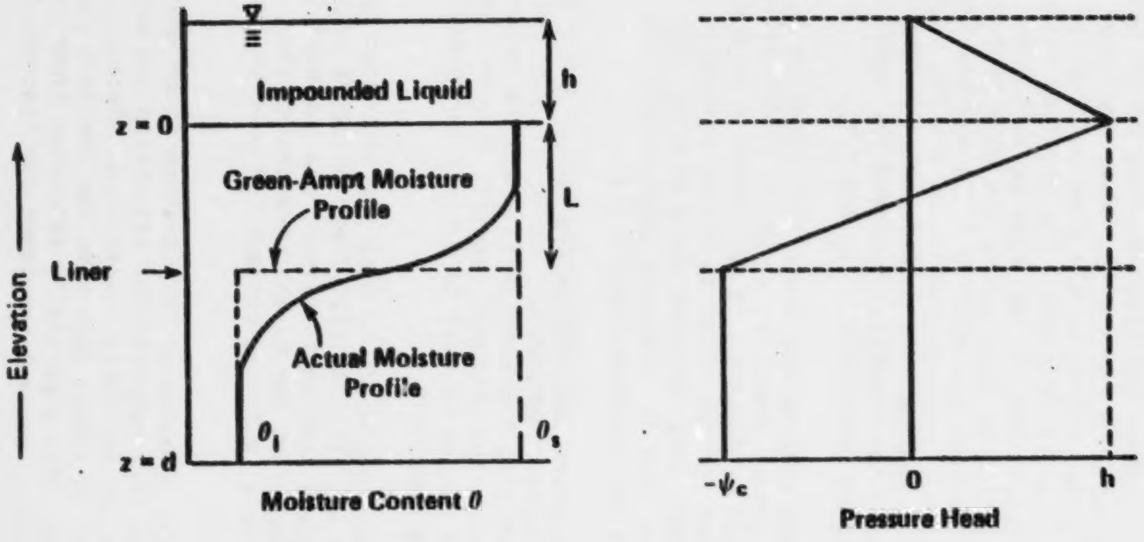
The transit time equations discussed above ignore both the initial, unsaturated nature of the liner and infiltration dynamics. Green and Ampt (1911) derived a simple model to describe the infiltration process that has been proposed as a method to assess liner reliability (U.S. EPA, 1984, Appendix A). A similar approach was also analyzed by McWhorter and Nelson (1979) and discussed by EPA (U.S. EPA, 1983).

The Green-Ampt model (1911) describes moisture movement in unsaturated soil during ponded infiltration by assuming a sharp, saturated wetting front moving down the soil column as a square wave. The model is schematically shown in Figures 8-4a and 8-4b. Figure 8-4a shows a schematic of the liner flow domain and also indicates the variation of moisture content with depth in the liner. Figure 8-4b graphically shows the corresponding pressure potential profile. Above the wetting front, the soil is fully saturated, with a moisture content, θ_s, while below the wetting front the moisture content is equal to its initial value, θ_i. At a given time, t, following establishment of the ponded leachate head on top of the liner, the wetting front will have moved down a certain distance, L. Saturated flow analysis may be applied to the saturated zone above the wetting front to determine the Darcian velocity or flux as given by Equation (8.3). The pressure head or capillary suction potential below the wetting front is $-\Psi_c$, due to the partial, initial saturation of the clay liner. At the top of the liner ($z = 0$), the pressure is given by the impoundment height ($\Psi = h$); at the wetting front ($z = L$), the potential is given by the suction ($\Psi = -\Psi_c$). Thus, when these boundary conditions across the saturated zone are incorporated, the Darcian velocity or flux (q) is given by assuming a linear potential gradient:

$$v = q = K_s \left(\frac{h+\Psi_c}{L} + 1\right) . \qquad (8.10)$$

In Equation (8.10) the wetted depth, L, is variable with time and is related to the flux, q, by conservation of pore fluid volume:

$$q = (\theta_s - \theta_i) \frac{dL}{dt} . \qquad (8.11)$$



a) Moisture Content Profile in Liner

b) Pressure Potential Profile

Figure 8-4. Green-Ampt infiltration model.

The time required for the wetting front to reach a depth of L is obtained by combining Equations (8.10) and (8.11) and integrating:

$$t = \frac{\theta_s - \theta_i}{K_s} \left[L - (h + \Psi_c) \ln \left(1 + \frac{L}{h+\Psi_c}\right)\right] . \qquad (8.12)$$

Thus, Equation (8.12) predicts the time required for an unsaturated clay liner to become saturated to a depth of L. When the liner thickness, d, is substituted for the wetted depth, L, the time required for the wetting front to reach the liner bottom is obtained. The required liner thickness for a certain specified transit time, t, can also be calculated. The seepage flux at the liner bottom is zero before the wetting front reaches the bottom. The seepage flux when the wetting front reaches the bottom may be obtained by substituting the liner thickness, d, for the wetted depth, L, in Equation (8.10). This seepage flux relationship is identical with that in the modified transit time equation, provided the same values are used for the capillary suction potentials, $- h_d$ and $- \Psi_c$.

The "transit time" predicted by the Green-Ampt model is not necessarily a true transit time. The solute transport equation is not solved in this approach, and the time given by Equation (8.12), after d is substituted for L, is only the time required for the liner to become saturated. If the clay liner is initially saturated, this equation would predict zero transit time. However, for a liner that is initially unsaturated, the saturation time will be equivalent to the leachate transit time.

Because this approach gives the seepage flux after liner saturation and the time required to achieve saturation, it can be used only with performance criteria prescribing the seepage flux and the saturation time. This model does not determine the leachate chemical flux or concentration at the liner bottom, so it cannot be used to predict performance with criteria based on leachate chemicals.

To use Equation (8.12), one needs to know the saturated hydraulic conductivity of the liner material, initial liner moisture content, saturation moisture content, liquid head on the liner, and the capillary suction potential below the wetting front due to the unsaturated clay. Capillary potential depends upon initial moisture content and must be determined for each clay liner material.

This model handles the dynamics of infiltration in an approximate manner. It determines the time required for saturation and would be applicable to clay liners that are initially unsaturated. Because it does not consider solute transport, it cannot determine the leachate concentrations and leachate transit times. For a partially saturated liner, the time required to achieve saturation may not represent the leachate transit time.

8.3.4 Transient Linearized Infiltration Equation

The general equation for flow through an unsaturated media is highly nonlinear because the hydraulic conductivity and capillary suction potential depend on the moisture content of the soil. Because the top boundary condition for this equation is expressed as a positive pressure (due to the ponded liquid) and the bottom boundary and initial liner conditions are given by initial pressure head, it is convenient to write Equation (8.1) in terms of pressure head (ψ) (Bear, 1979):

$$\frac{d\theta}{d\psi} \cdot \frac{\partial\psi}{\partial t} = \frac{\partial(K\frac{\partial\psi}{\partial z})}{\partial z} - \frac{dK}{d\psi} \cdot \frac{\partial\psi}{\partial z} \qquad (8.13)$$

where

K = effective hydraulic conductivity, depends upon ψ, and is equal to $K_s\ k_r$

θ = moisture content and also depends on ψ

z = vertical coordinate expressed positive downward

When both sides are multiplied by ($d\psi/d\theta$):

$$\frac{\partial\psi}{\partial t} = \frac{d\psi}{d\theta} \frac{\partial(K\frac{\partial\psi}{\partial z})}{\partial z} - \frac{d\psi}{d\theta} \frac{dK}{d\psi} \frac{\partial\psi}{\partial z} . \qquad (8.14)$$

Defining $D^* = K\,(d\psi/d\theta)$ and $K^* = dK/d\theta$ and substituting in Equation (14) (Richards, 1931; Philips, 1957) obtains:

$$\frac{\partial\psi}{\partial t} = D^* \frac{\partial^2\psi}{\partial z^2} - K^* \frac{\partial\psi}{\partial z} . \qquad (8.15)$$

Equation (8.15) is the linearized form of the nonlinear infiltration equation, and its analytical solution may be obtained by assuming D^* and K^* to be constant for a given clay liner. Moore (1980) has previously recommended this technique for impoundment liner evaluation. To obtain the analytical solution, one must assume the clay liner to be a semi-infinite medium; i.e., one must assume that the liner extends to an infinite depth and that its pressure head at the infinite boundary remains unchanged from its initial value. At the top of the liner ($z = 0$), pressure head is equal to the depth of ponded liquid ($\psi = h$); initial ($t = 0$) pressure head throughout the liner is equal to ψ_i; and at the bottom of the liner ($z = \infty$), pressure head remains at its initial value ($\psi = \psi_i$). When these boundary and initial conditions are applied, the analytical solution may be obtained as follows (Bear, 1979):

$$\psi = \psi_i + \frac{h-\psi_i}{2} \left[\operatorname{erfc} \left(\frac{z-K^*t}{2\sqrt{D^*t}} \right) + \exp \left(\frac{K^*z}{D^*} \right) \operatorname{erfc} \left(\frac{z+K^*t}{2\sqrt{D^*t}} \right) \right] \quad (8.16)$$

where

erfc = complementary error function.

Solutions to Equation (8.16) are presented graphically in Figure 8-5 with the liner thickness, d, replacing z. Unlike the transit time equations and the Green-Ampt model, the transient linearized solution results in a continuous profile of soil moisture that is physically more accurate. There is no sudden increase in moisture content or reduction in the suction potential at the liner bottom. Rather, moisture content gradually increases from the initial value to saturation as the wetting front advances. Due to the semi-infinite boundary condition employed, the solution is accurate until the time when the moisture content at the liner bottom begins to rise.

A model based on the transient linearized infiltration equation solves the flow equation but does not consider the solute transport equation. Like the Green-Ampt model, this approach is most suitable when the clay liner is initially unsaturated. The transit time calculated with Equation (8.16) is the time required to saturate the clay liner. When the initial moisture content is high, the wetting front advances faster than the corresponding leachate front; in such situations, the transit time predicted by Equation (8.16) does not reflect the actual leachate chemical transit time.

For initially unsaturated clay liners, the leachate transit time may be approximated by the time required to achieve liner saturation. Because the moisture content as predicted by this linearized model increases gradually, there is no sudden or well-defined breakthrough of the saturation front. Therefore, it is necessary to define the breakthrough of the front in terms of relative changes in moisture content or pressure. For example, a common choice is the point where suction potential is reduced by 50 percent; but significant amounts of liquid may have seeped through before the capillary suction potential is reduced by 50 percent. A conservative value for the quantity $(\psi - \psi_i)/(h - \psi_i)$, such as 0.1, may be used arbitrarily to represent the saturation breakthrough. Figure 8-5 may then be used to determine the required liner thickness. An iterative technique must be used by assuming a thickness, d, computing D^*/K^*d, and finding K^*t/d on the bottom axis. Time, t, is then computed, and a new value of d is assumed depending on whether t is less than or greater than the design life.

To use the transient linearized infiltration equation, one must know the initial moisture content of the clay liner and corresponding negative pressure head, ψ_i; the depth of impounded liquid on top of the liner, h; and the parameters D^* and K^*. The variation of hydraulic conductivity, K, and pressure potential, ψ, with clay moisture content, Θ, must be known to determine the last two parameters. These relationships are usually nonlinear;

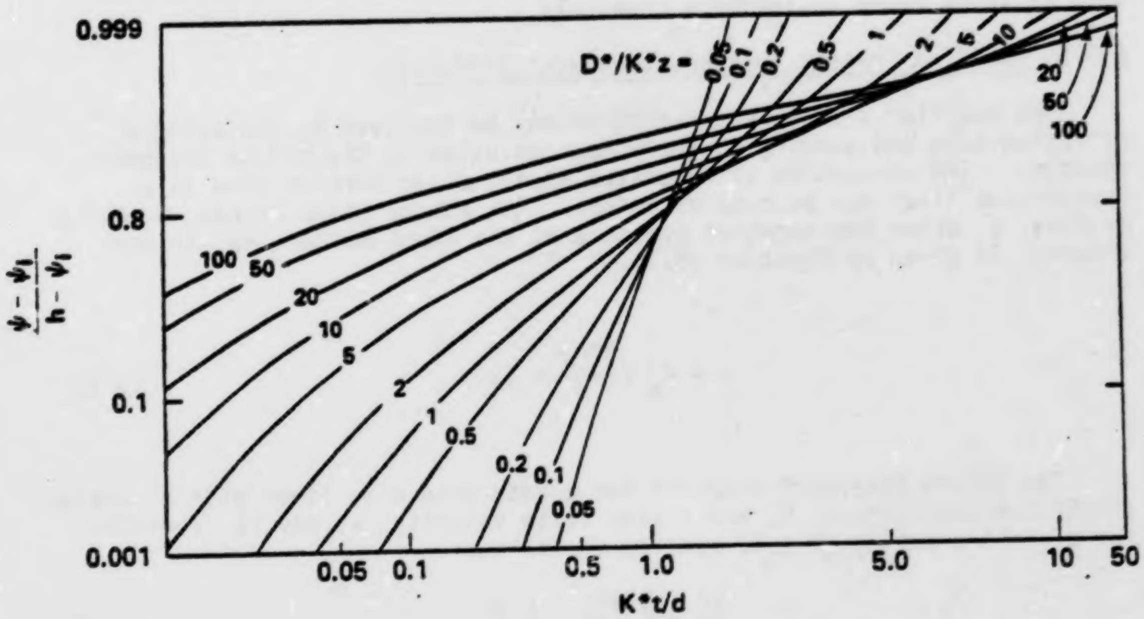


Figure 8-5. Graphical solution to equation (8.16)—linearized infiltration model.

therefore, it is difficult to assign reasonable values to D* and K* as defined in Equation (8.15), even after these relationships are obtained from laboratory measurements.

This model determines the time required for the clay liner to become saturated, and it accounts for the dynamics of infiltration processes. The seepage flux, before the moisture content at the bottom of liner begins to rise, is negligible. After liner saturation, the steady state seepage flux may be obtained by an equation similar to (8.10) or (8.7). Like the Green-Ampt wetting front model, this approach does not determine the leachate chemical flux or concentration and therefore cannot be used with performance criteria based on leachate chemicals.

8.3.5 Modified Transit Time Equation with Diffusion

The modified transit time equation may be improved by including a diffusion term and seeking an analytical solution to the solute transport equation. The assumption of saturated steady state Darcian flow in a homogeneous liner may be made as before. The steady state Darcian velocity or flux, v, after the negative pressure at the liner bottom, $-h_d$ is considered, is given by Equation (8.7):

$$v = K_s \left(\frac{h+h_d}{d} + 1\right) . \tag{8.7}$$

The solute transport equation for a saturated clay liner with a constant dispersion coefficient, D, and steady state velocity, v, may be rewritten for a noninteracting (R = 1) species as follows:

$$\frac{\partial C}{\partial t} = \frac{D}{\phi}\frac{\partial^2 C}{\partial z^2} - \frac{v}{\phi}\frac{\partial C}{\partial z} . \tag{8.17}$$

Equation (8.17) is very similar to Equation (8.15). To obtain an analytical solution to Equation (8.17), one must again assume that the clay liner extends to an infinite depth and its concentration at the infinite boundary remains unchanged from its initial value. The initial and boundary conditions are as follows: at the top of the liner (z = 0), the leachate concentration is C_o, ($C = C_o$); initially (t = 0), concentration throughout the liner is zero (C = 0); and at the bottom of the liner ($z = \infty$), concentration remains unchanged (C = 0). The analytical solution for Equation (8.17) may be written similarly to Equation (8.16) (Bear, 1979; Ogata and Banks, 1961; Cleary and Ungs, 1978):

$$C = \frac{C_o}{2} \quad \mathrm{erfc}\left[\frac{z - \frac{vt}{\phi}}{2\sqrt{\frac{Dt}{\phi}}}\right] + \exp\left(\frac{vz}{D}\right) \mathrm{erfc}\left[\frac{z + \frac{vt}{\phi}}{2\sqrt{\frac{Dt}{\phi}}}\right] . \tag{8.18}$$

Equation (8.18) describes concentration profiles of the leachate chemical species with time. At a given location, the concentration rises gradually from zero to the leachate concentration at the top of liner. Because there is no explicit or sudden breakthrough of the leachate front at the liner bottom, it is necessary to define leachate breakthrough in terms of a relative change in concentration (C/C_o) at the liner bottom. For example, an average value of 0.5 may be chosen; however, significant amounts of chemicals may escape the liner before the relative concentration at the liner bottom reaches a value of 0.5. A conservative value of $C/C_o = 0.1$ may be used arbitrarily to represent the leachate chemical breakthrough. Figure 8-6 shows a graphical solution to Equation (8.18).

An iterative technique similar to that discussed in Section 8.3.4 above may be used to obtain the required liner thickness for a specified leachate transit time. Equation (8.18) predicts the leachate concentration at the liner bottom with time. The steady state seepage flux is given by Equation (8.7). Again, the leachate chemical flux is obtained by multiplying the seepage flux by the leachate concentration. The leachate chemical flux at the liner bottom is negligible before the concentration there begins to rise.

To use the modified transit time equation with diffusion, one must know the saturated hydraulic conductivity and effective porosity of the liner material; the depth of impounded liquid; the capillary suction potential at the liner bottom; and the effective dispersion coefficient for the leachate chemical species. For low-permeability clays, the dispersion coefficient may be approximated by the molecular diffusion coefficient of the leachate species in water. This equation is simple to use. It assumes a saturated liner medium but accounts for the effect of suction potential at the liner bottom and the contributions of both advection and molecular diffusion to the solute transport process.

8.3.6 Numerical Solutions

The various models discussed in previous sections attempt to obtain simplified solutions to Equations (8.1) and/or (8.2), which describe flow and solute transport in saturated and unsaturated media. Section 8.3.4 represents an attempt to linearize the flow equation for unsaturated media by introducing simplifying assumptions. Although the flow equation is relatively independent of the transport equation, the solute transport equation is coupled with the flow equation due to the advective contribution of flow to solute transport. For this reason, linearization of the solute transport equation is especially difficult. Section 8.3.5 describes an analytical solution for flow and solute transport in saturated media. Such a solution is not possible for unsaturated media due to nonlinearities involved in the governing equations.

Numerical methods can be used to describe accurately the movement of the saturation front and solute transport in unsaturated clays. Computer programs are available to solve the generalized unsaturated flow and solute transport problem in one or two dimensions. They generally employ finite

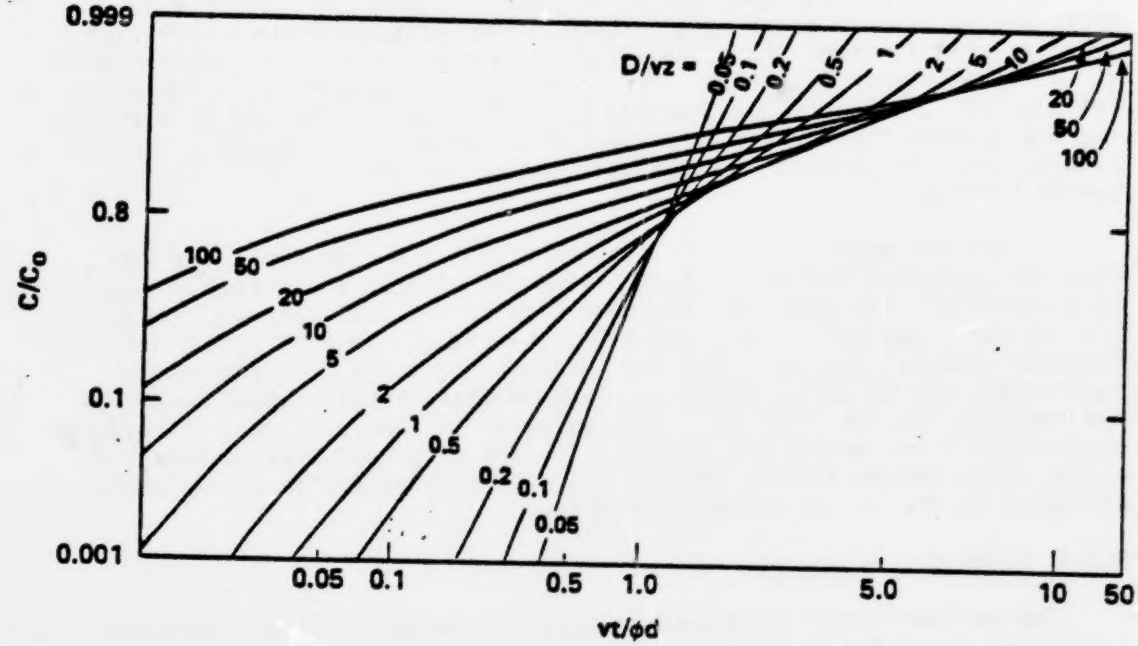


Figure 8-6. Graphical solution to equation (8.18)—transit time equation with diffusion.

difference or finite element numerical techniques and require data regarding the variation of hydraulic conductivity and moisture potential with moisture content in clay and underlying site soil. To use the numerical methods, one should be familiar with both the numerical techniques and the requirements and limitations of the specific program used.

Numerical methods reduce governing Equations (8.1) and (8.2) to approximating algebraic equations that are then solved through implicit linear algebra techniques. Finite difference techniques approximate the partial derivatives in the governing equations by using finite difference equations (Pinder and Bredehoeft, 1968; Mercer and Faust, 1980). The finite difference equations are solved for each node on an arbitrary grid through an iterative procedure. Compared to finite element codes, finite difference codes are simple, are relatively easy to program, and require less computer time for data manipulation. Finite element approximations incorporate a finite series of basis (or shape) functions and associated time-dependent coefficients to describe the dependent variables. The series are then input to the governing equations and the resulting error or residual is minimized (Pinder, 1973; van Genuchten, 1978). Finite element codes are more complex, require more input data, and are more expensive to execute. However, the finite element codes are more flexible and can handle irregular flow geometries and boundary conditions. Because no irregular flow geometries or boundary conditions are involved, simple one-dimensional finite difference codes may be adequate to describe the vertical flow and solute transport through a clay liner system.

Several computer codes are available to solve the unsaturated flow and/or solute transport problem in one or two dimensions. A complete list may be obtained from a computerized data base called the Model Annotation Retrieval System, maintained by the International Groundwater Modeling Center at the Holcomb Research Institute of Butler University. A partial list of some of the models and their characteristics is given in Appendix B. A comprehensive list is also given by Bachmat et al. (1980).

8.3.6.1 Numerical Solution of the Flow Equation--

The solution to Equation (8.1), the general flow equation, for an unsaturated clay liner yields moisture content profiles in a clay liner with time and also the rate of seepage flux through the liner at different locations as a function of time. Notable among the available programs to solve the unsaturated flow equation in a liner system is a finite difference program, "SOILINER," developed by Goode and Smith (in U.S. EPA, 1984). This program was specifically written for a clay liner system and is described in detail in their report. This program considers vertical one-dimensional flow in the flow domain shown in Figure 8-1 and solves the equation in the form given in Equation (8.13), which incorporates suction potential (Ψ). The following sections describe the input data required by the SOILINER program.

8.3.6.1.2 Grid Specifications--The vertical linear flow domain may be divided by a certain number of node points. The number of node points and spacing between the node points (element length) must be specified. The spacing may be uniform or graded. Alternatively, the coordinate for each node point may be specified for maximum flexibility in node spacing.

8.3.6.1.3 Clay and Soil Properties--Saturated hydraulic conductivity and porosity for each element in the flow domain may be specified. This allows maximum flexibility to account for heterogeneous layers in the flow domain. Typically, two or three layers may be included to represent the clay liner, underlying site soil, and intermediate (optional) leak detection zone. In addition, functional relationships for each different type of material must be provided to determine the relative hydraulic conductivity (k_r), moisture content (Θ), and soil-specific moisture capacity ($d\Theta/d\Psi$) as a function of capillary suction potential (Ψ). These relationships are obtained by least square fitting the experimental data by a continuous functional form. The program has included functional relationships for three different soil types (clay and two different sandy soils), and more relationships can easily be included.

8.3.6.1.4 Initial Conditions--Because the program solves an unsteady state equation, initial conditions must be specified. For example, initial distribution of the pressure head for the flow domain may be specified. The clay liner may be assumed to have a constant initial moisture content and corresponding uniform initial suction potential. The underlying site soil may be assumed to be at static equilibrium with respect to the water table.

8.3.6.1.5 Boundary Conditions--Also needed are the boundary conditions at the top and bottom of the flow domain. The boundary condition at the top is due to the impounded liquid depth on top of the liner. The pressure head at the top is thus equal to the depth of the free liquid. The boundary condition at the bottom is critical because it contributes to the overall potential gradient across the liner and flow domain. If the underlying site soil above the water table is included in the flow domain, the bottom boundary potential will be zero, corresponding to the free liquid surface of the water table. If only the clay liner is considered in the flow domain, the suction potential at the bottom of the liner due to the underlying site soil needs to be estimated. If it is assumed that the liner bottom is ventilated and open to the atmospheric air, the suction potential at the liner bottom will remain at the initial level until the moisture content at the bottom begins to rise or the saturation front reaches the bottom.

Similar input data will be required by all the numerical codes solving the unsaturated flow equation. To run such a program, a user often also needs to specify the maximum time step the program should use and the final end time. In addition, those times must be specified to obtain moisture profiles and flux at intermediate times. The output from such a flow problem typically includes distribution of suction potential, moisture content, flow velocity, and seepage flux at each node (or specified output nodes) in the flow domain.

Most of the programs for unsaturated flow were developed for a generalized unsaturated flow situation (typically infiltration into soil) and thus may require some effort to be adapted to the present clay liner problem. However, the SOILINER program is written specifically for flow through an unsaturated clay liner.

The solution of the unsaturated flow equation describes the variation of moisture profiles and seepage flux in the liner with time. Like the linearized flow model, the numerical solution predicts a gradual increase in the moisture content and reduction in the suction potential. Thus, there is no sudden or explicit breakthrough of the saturation front at the liner bottom. As before, such a concept may be defined on the basis of the relative reduction in the suction potential at the liner bottom. A conservative value for the relative reduction $[(\Psi-\Psi_i)/(h-\Psi_i)]$, such as 0.1, may be used to represent the saturation breakthrough. Thus, the user can predict the time required for the liner to become saturated or for the saturation front to reach the bottom.

However, the numerical solution to the flow equation does not address the transport of leachate chemicals. It does not determine the leachate chemical concentration profiles and the flux with time; therefore, it cannot predict the true leachate transit time. By arguments similar to those presented for the Green-Ampt model and the linearized flow model, this approach is suitable for clay liners that initially are predominantly unsaturated. Where the clay's initial moisture content is high, the saturated wetting front may advance faster than the front representing the leachate chemicals. Where the liner is initially saturated, the unsaturated flow equation is not applicable.

8.3.6.2 Numerical Solution of the Solute Transport Equation--

It is necessary to solve Equation (8.2), the solute transport equation, to describe the transport of leachate chemicals in an unsaturated clay liner system. As discussed previously, the unsaturated solute transport equation is coupled with the unsaturated flow equation due to advective contribution of the seepage flux to the solute transport (the term involving Darcian velocity, v, in Equation (8.2)) as well as due to the variation of moisture content with time that is determined from the flow equation. For this reason, the solute transport equation and the flow equation must be solved simultaneously. Alternatively, the variation of velocity and moisture content at each node with time may be stored in a separate data file while the flow equation is solved. This data file may then be used as an input to the solute transport program, in addition to other input parameters for the solute transport program, to obtain solute concentration profiles and solute fluxes with time.

Almost all of the available computer programs for solving the unsaturated solute transport problem require the solution of the flow problem first because they do not solve both equations simultaneously. Thus, it is necessary to store the velocity and moisture content values at each node for all time steps in a separate file. Solute transport usually includes diffusional as well as convective elements. The dispersion coefficient includes the effect of both molecular diffusion and hydrodynamic dispersion. For clay liners with very low permeability, the molecular diffusion may be expected to dominate the dispersion process. Since different chemical species in the leachate may have differing dispersion and attenuation properties, they may migrate differently in the flow domain. Therefore, the solute transport equation must be solved specifically for the chemical species of interest. Different solute transport equations may be written for specific chemicals by using respective dispersion and retardation coefficients; these equations may be solved simultaneously to obtain concentration profiles and chemical fluxes for each chemical. Since the bulk flow velocity and the moisture content profiles would be the same for any number of chemicals in the leachate, the flow problem needs to be solved only once, irrespective of the number of chemical species in the leachate.

The input data required to solve the solute transport problem are described below:

8.3.6.2.1 Grid Specifications--These are the same data required for the flow problem (Section 8.3.6.1).

8.3.6.2.2 Clay and Soil Properties--Since the flow equation must be solved along with the solute transport, all the above-mentioned physical data are needed for the companion flow problem program. In addition, for the solute transport problem the dispersion coefficient, D, must be specified for all elements. If the hydrodynamic dispersion is ignored, the dispersion coefficient would simply be the molecular diffusion coefficient of the chemical species of interest in water and would be reasonably constant for the entire flow domain. If the hydrodynamic dispersion coefficient is included, the functional relationship between the hydrodynamic dispersion coefficient, D_h, and Darcian flow velocity must be specified as in:

$$D_h = \frac{\alpha_L}{\phi} v \qquad (8.19)$$

where

α_L = longitudinal dispersivity.

Since the flow is considered to be one-dimensional, the transverse dispersivity values are not required and may be taken as zero. The longitudinal dispersivity, α_L, depends upon the type of flow medium and may have different values for each heterogeneous layer in the flow domain such as clay and underlying site soil. Since the hydrodynamic dispersion coefficient depends upon flow velocity, it needs to be evaluated for each time step for all elements. The overall dispersion coefficient, D, is obtained by adding together the molecular diffusion coefficient and the hydrodynamic dispersion coefficient.

Also needed is the retardation coefficient, R, for all elements, to account for the leachate attenuation capacity of the clay and soil. The retardation coefficient is specific for a leachate chemical/soil combination and must be determined experimentally. Thus, the clay and the underlying site soil may have different attenuation capacities for a given chemical. The attenuation capacity usually slows down the migration of a chemical, therefore ignoring that the attenuation is likely to yield a conservative estimate of the required liner thickness. If the attenuation capacity of the soil/clay is ignored, the value of the retardation coefficient, R, is set to 1.

The flow equation is usually considered independent of the solute transport equation. One way in which leachate chemicals can affect the solution of the flow equation is by altering the permeability of clays and soils. Many pure organic solvents and inorganic acids and bases have a strong effect on the properties of the flow medium. Such effects, if known, need to be taken into account to solve the flow equation.

8.3.6.2.3 Initial Conditions--To solve the unsteady state solute transport equation, initial concentration values of the chemical species in the flow domain must be specified. Usually, the initial solute concentrations may be assumed to be zero.

8.3.2.6.4 Boundary Conditions--Like the flow problem, the boundary conditions at the top and bottom of the flow domain need to be specified. The concentration of chemical species at the top is the concentration in the impounded liquid, C_o. The concentration at the bottom is difficult to designate. Ideally, there is a matching chemical flux condition at the bottom to represent inflow and outflow. If the underlying site soil is included, the matching condition may be described by the dilution of the leachate seepage due to the aquifer flow rate. If this dilution is considered to be large enough, the concentration at the bottom boundary may be taken to be zero. This boundary condition is difficult to specify if the underlying site soil is not included in the flow domain. One alternative is to assume that the underlying site soil does not influence the leachate chemical flux; mathematically, the leachate chemical flux may be assumed to be the same in the bottom two elements:

$$\left(D\frac{\partial C}{\partial z} - vC\right)_n = \left(D\frac{\partial c}{\partial z} - vC\right)_{n-1} \qquad (8.20)$$

where

n = bottom element.

8.3.6.2.5 Darcian Velocity and Moisture Content Profiles--These parameters are obtained by solving the flow equation first. The moisture content and flux values at each node need to be saved for all time steps in an input file that is later recalled from the solute transport program. If the flow and solute transport equations are solved simultaneously, such saving of data will not be needed.

Similar data are needed for all codes solving the solute transport equation. The user also needs to specify the maximum allowable time step and the end time. For stability, it may be necessary to have smaller time steps than those used in the flow problem. If this is the case, a particular flow time step at a certain time may need to be subdivided for the solute transport problem. The velocity and moisture content values will then remain the same until the time is advanced by the flow step.

The typical output from a solute transport code will indicate the solute chemical concentrations and fluxes at all nodes for all times or for specified output times. Solving both the flow and the solute transport equations determines the moisture content profiles as well as the chemical concentration profiles with time along with the seepage fluxes and the chemical fluxes. At a given location, the concentration rises gradually from zero to the leachate concentration at the liner top. Thus, there is no sudden or explicit breakthrough of the leachate at the liner bottom. Therefore, breakthrough must be defined in terms of the relative change in concentration (C/C_o) at the liner bottom. As discussed before, a conservative value of $C/C_o = 0.1$ may be arbitrarily used to represent the leachate chemical breakthrough. Such a definition may be used to determine the time required for leachate chemical breakthrough.

Compared to other prediction methods, the numerical solutions of the flow and solute transport equations for unsaturated media provide a great deal of information. The numerical solutions provide an accurate description of the leachate migration, and they can be used with any set of performance criteria. However, numerical techniques are expensive to run and require expertise in the use of the computer models. Available programs to solve the flow equation outnumber those available for the solute transport problem. Although some of the programs are proprietary, two available flow programs in the public domain are SOILINER, which was developed by GCA and is written specifically for clay liners, and FEMWATER, which was developed by Oak Ridge National Laboratory (ORNL). ORNL also has a solute transport program, FEMWASTE, available in the public domain. In the present work, SOILINER was modified by incorporating a subroutine CONCAL to solve the solute transport equation simultaneously with the flow equation. A simple forward-time centered space difference scheme was used to solve the solute transport equation. Details of this modified program, LINERSOL, are given in Appendix C. In LINERSOL, the flow solution is first advanced by a flow time step. The current moisture content and velocity profiles are then used to advance the solute transport equation by the same flow time step. For stability and accuracy, it is often necessary to divide a flow time step into a smaller solute transport time step. The program LINERSOL can thus solve both the flow and solute transport equation simultaneously for a liner system.

8.4 COMPARISON OF DIFFERENT APPROACHES

This section compares each of the transit time prediction methods discussed above. To compare the methods under equal concitions, we first describe a "basic liner scenario" that specifies assumptions concerning initial site conditions and the leachate flow domain. Because the methods make different assumptions, different transit time predictions may be expected both where the initial moisture content of the liner varies and under different performance criteria. Three different initial moisture contents and two different performance criteria are considered in the following analysis.

8.4.1 Basic Assumptions

The leachate flow domain is assumed to consist of 2 m of compacted clay liner placed on top of underlying site soil. The depth of this site soil is assumed to be 5 m above the water table. The water table is assumed to be stable with time and the leachate head on top of the liner is assumed to be 30 cm (this is the maximum allowed by current regulations).

The clay liner is assumed to have an effective porosity of 0.495 and a saturated hydraulic conductivity of 1×10^{-7} cm/s. The underlying site soil is considered to have an effective porosity of 0.287 and a saturated hydraulic conductivity of 1×10^{-2} cm/s. The relationships among the moisture content, hydraulic conductivity, and capillary suction potential for the clay and site soil are assumed to be those given by Figure 8-7 (U.S. EPA, 1984). For a leachate chemical species, it is assumed that the dispersion coefficient is dominated by molecular diffusion and that the molecular diffusion coefficient, D, in water is 1×10^{-6} cm²/s. The attenuation capacity of the clay soil for this chemical species is assumed to be negligible by setting the attenuation coefficient, R, equal to 1.

The site soil is initially assumed to be in static equilibrium with the water table. Thus, the initial suction potential varies linearly with the depth, and at the top of the site soil the suction potential is -500 cm. Because the site soil has a very high permeability relative to the clay liner, the flow through the liner may be expected to have only a small effect on the suction potential profile in the site soil.

The liner simulations with each of the prediction methods are carried out based on three different values for the initial moisture content of the clay liner: 50 percent, 80 percent, and 95 percent of saturation. The initial suction potential at the time of placement is assumed to be constant throughout the clay liner and is determined by the initial moisture content as illustrated in Figure 8-7. The following suction potentials are associated with the three values of initial moisture content:

Saturation	Suction potential (ψ)
50 percent	-500 cm
80 percent	-57.38 cm
95 percent	-14.82 cm

The clay liner is assumed to be exposed to a constant leachate head of 30 cm on top of the liner immediately after placement in the field.

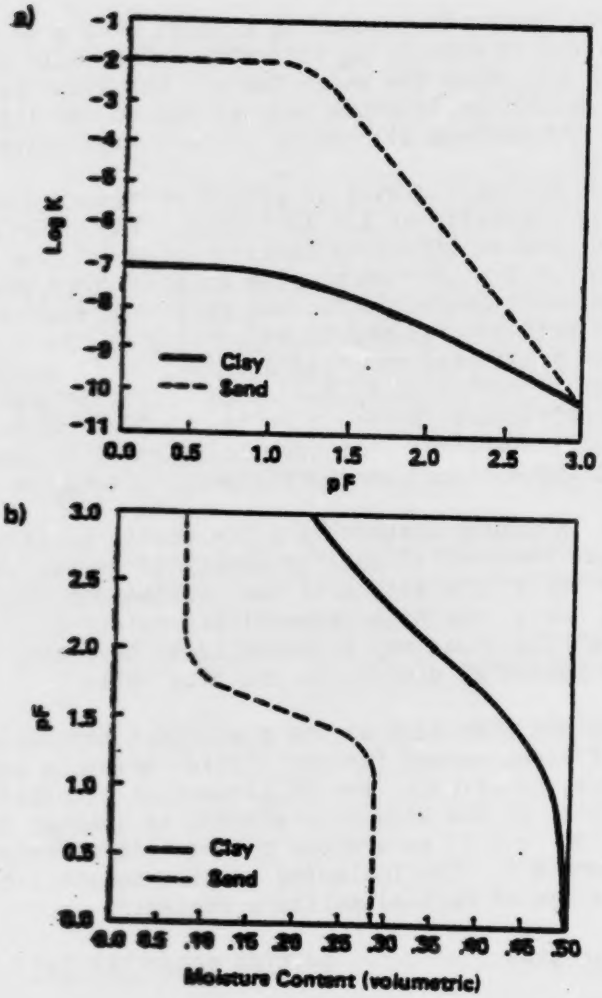


Figure 8-7. Soil properties of hypothetical clay (solid line) and sand (dashed line) for liner design. a) Unsaturated hydraulic conductivity (cm/s) as a function of capillary pressure (pF = log(−ψ), ψ in cm) b) Moisture content as a function of capillary pressure. (EPA, 1984)

Based on the scenario described, two different performance criteria were applied to determine the breakthrough time predicted by each method. One performance criterion is based on the flow equation and the advancement of the saturation front. The second criterion is based on solute transport in the liner and the advancement of the leachate chemical front. Table 8-2 summarizes the results of the liner simulations for each method under the two performance criteria. The following sections present the assumptions and calculations for each simulation.

8.4.2 Comparison With Performance Criteria Based on Saturation Front

Performance criteria based on movement of the saturation front may be defined in several ways. For example, transit time may be defined as the time taken for the liner to become saturated to achieve a steady state moisture content profile, or for the relative reduction in the suction at the liner bottom to be equal to or greater than 0.1 (i.e., $(\Psi-\Psi_i)/(h-\Psi_i) \geq 0.1$). Where applicable, each of the methods discussed in previous sections was used to determine the transit time based on the saturation front criteria.

8.4.2.1 Simple Transit Time Equation--

This approach assumes a saturated liner and thus cannot be used with the above criteria. With this approach, the time required for saturation equals zero.

8.4.2.2 Modified Transit Time Equation--

This approach also assumes a saturated liner so the time required to saturate the liner is zero.

8.4.2.3 Green-Ampt Wetting Front Model--

Equation (8.12) is used to obtain the required time to saturate the liner. The depth of saturation of the liner, L, is replaced by the liner thickness, 200 cm. The suction potential in front of the wetting front, $-\Psi_c$, depends upon the initial moisture content in clay liner. The saturated hydraulic conductivity of the clay liner is 1×10^{-7} cm/s, and the leachate head on top of the liner is 30 cm. Table 8-3 shows the parameters and the predicted transit times assuming three values for initial moisture content of the liner.

Under actual conditions, the time required to saturate the liner should decrease with increasing initial moisture content. However, due to the use of approximate nonlinear suction potential characteristics and also the arbitrary nature of specifying the suction potential in front of the wetting front, the calculated results are different than expected. This indicates that a good choice of the suction potential is necessary to obtain consistent results.

8.4.2.4 Transient Linearized Infiltration Model--

With this method, transit time is obtained from Equation (8.16) or Figure 8-5, with the liner thickness, d, which is 200 cm, substituted for z. The parameters D* and K* are determined at an average liner moisture

TABLE 8-2. RESULTS OF COMPARING TRANSIT TIME PREDICTION METHODS

	Transit time prediction (yr)					
	Performance criteria based on the saturation front			Performance criteria based on solute transport		
	Initial liner saturation			Initial liner saturation		
Transit time prediction method	50%	80%	95%	50%	80%	95%
1 Simple transit time equation[a]	0	0	0	27.3	27.3	27.3
2 Modified transit time equation[a]	0	0	0	8.6	8.6	8.6
3 Green-Ampt wetting front model	2.4	3.01	1.0	NA	NA	NA
4 Transient linearized infiltration model	19.7	4.69	2.0	NA	NA	NA
5 Modified transit time equation with diffusion[a]	0	0	0	6.0	6.0	6.0
6 Numerical solution of the flow equation	10	5	2.5	NA	NA	NA
7 Numerical solution of the solute transport equation	NA	NA	NA	11.5	16.0	17.5

NA = not applicable.

[a] This approach assumes a saturated liner; thus, initial moisture content does not affect the transit time prediction.

TABLE 8-3. TRANSIT TIME ESTIMATION USING THE GREEN-AMPT WETTING FRONT MODEL

	Parameters for Green-Ampt model		
Initial liner saturation (%)	Moisture content by volume	Suction potential $-\Psi_c$ (cm)	Transit time (yr)
50	0.248	-500.00	2.38
80	0.396	-57.38	3.01
95	0.47	-14.82	0.97

content, $(\theta_i + \theta_s)/2$. Table 8-4 shows the parameters and the predicted transit times assuming three values for the liner's initial moisture content.

Unlike the Green-Ampt model, the linearized flow model consistently predicts longer transit times as the initial liner moisture content is decreased. This indicates that the model describes the dynamics of infiltration better than the Green-Ampt equation. The predicted transit times are consistently longer than the corresponding Green-Ampt values.

8.4.2.5 Modified Transit Time Equation With Diffusion--

This approach considers the liner to be saturated, so the time required for saturation is zero.

8.4.2.6 Numerical Solution of the Flow Equation--

The computer program SOILINER was used to determine the transit time based on the saturation front. Since the site soil was included in the simulation, which provided a suction potential of about -500 cm at the liner bottom, the moisture content profiles in the liner tend to achieve steady state distribution after a certain time period. Thus, where the initial moisture content was higher (80 percent and 90 percent), some drainage in the lower portion of the liner was predicted. The seepage flux also achieves a steady state value. Before the steady state moisture profile is achieved, the seepage flux is low and reaches a higher steady state value while the moisture profile is established. Therefore, the time required to achieve this steady state moisture content profile was considered to be the transit time in these numerical solutions. Table 8-5 shows the predicted transit times for the three values of initial moisture content.

The numerical results correctly indicate that the time required to achieve saturation decreases as the liner moisture content increases. This time would approach zero if the initial moisture content increased to the saturation value. The linearized flow model shows better agreement with numerical results at higher initial moisture content because the assumption of constant D* and K* is more realistic when the change in moisture content of the liner is smaller.

8.4.2.7 Numerical Solution of the Solute Transport Equation--

This solution is not required for performance criteria based upon the saturation front alone.

8.4.3 Comparison Using Performance Criteria Based on Solute Transport

Performance criteria based on solute transport may be chosen, for example, to define transit time as the time required for the relative concentration of a chemical species at the liner bottom to achieve a value greater than or equal to 0.1 ($C/C_o \geq 0.1$). The steady state concentration of the leachate species throughout the liner after a sufficiently long time will be equal to its concentration in the leachate on top of the liner. This concentration, C_o, is assumed to be constant with time. Prediction methods are compared for three initial liner moisture contents of 50 percent, 80 percent, and 95 percent of saturation based on the above-defined perform-

TABLE 8-4. TRANSIT TIME ESTIMATION USING THE TRANSIENT LINEARIZED INFILTRATION MODEL

Initial liner saturation (%)	Moisture content by volume	Initial suction potential (cm)	Average moisture content	Average suction potential (cm)	Average K (cm/s)	$\frac{d\psi}{d\theta}$ (cm)	$\frac{dK}{d\theta}$ (cm/s)	D^* (cm/s)	K^* (cm/s)	$\frac{D^*}{K^*d}$	$\frac{K^*t}{d}$	Transit time (yr)
50	0.248	-500.00	0.3713	-80.22	5.041×10^{-9}	-1,066.47	-1.126×10^{-7}	5.376×10^{-6}	1.126×10^{-7}	0.2387	0.35	19.7
80	0.396	-57.38	0.4455	-26.21	2.777×10^{-8}	-498.85	-6.757×10^{-7}	1.385×10^{-6}	6.757×10^{-7}	0.1025	0.5	4.69
95	0.47	-14.82	0.483	-9.46	7.0×10^{-8}	-444.33	-1.745×10^{-6}	3.110×10^{-6}	1.745×10^{-6}	0.0891	0.55	2.0

TABLE 8-5. TRANSIT TIME ESTIMATION USING A NUMERICAL SOLUTION TO THE FLOW EQUATION

Initial liner saturation (%)	Moisture content by volume	Transit time[a] (yr)
50	0.248	10.0
80	0.396	5.0
95	0.470	2.5

[a] Transit time equals the time required to achieve a steady state moisture profile.

ance criteria. The dispersion coefficient, D, of the chemical species is assumed to be 1.0×10^{-6} cm²/s, and the attenuation coefficient, R, is taken to be 1.

8.4.3.1 Simple Transit Time Equation--

The required time is given by Equation (8.5). Because this equation assumes that the liner is saturated, the initial moisture content does not affect the predicted transit time. When the following values are substituted in Equation (8.5), the estimated transit time is 27.3 years:

Liner thickness: $d = 200$ cm

Liquid head: $h = 30$ cm

Saturated hydraulic conductivity: $K_s = 1.0 \times 10^{-7}$ cm/s

Porosity: $\phi = 0.495$.

8.4.3.2 Modified Transit Time Equation--

The required time is given by Equation (8.8). This equation also assumes the liner to be saturated, so the initial moisture content does not affect the predicted transit time. Since the liner is considered to be underlain by 5 m of site soil above the water table, the suction potential at the liner bottom, $-h_d$, is considered to be -500 cm. When this value, and other parameters as above, are substituted, the predicted transit time is 8.6 years.

8.4.3.3 Green-Ampt Wetting Front Model--

This approach does not consider the solute transport equation and hence cannot determine the time required for the above criteria. For highly unsaturated liners, the required solute transport time may be approximated by the saturation front transit time and may be calculated as in the previous section.

8.4.3.4 Transient Linearized Infiltration Model--

Like the Green-Ampt model, this approach also does not consider the solute transport equation and cannot determine the time required for the above criteria. For highly unsaturated liners, the time required for the solute transport may be approximated by that required by the saturation front and may be determined as before.

8.4.3.5 Modified Transit Time Equation With Diffusion--

The required time for the solute chemical breakthrough may be obtained from Equation (8.18) or Figure 8-6 by substituting liner thickness, d, for z. This approach also assumes a saturated liner, so the initial moisture content of the liner will not affect the predicted transit time. The seepage flux or Darcian velocity is first obtained as given by Equation (8.7). The following values are substituted:

Saturated hydraulic conductivity: $K_s = 1.0 \times 10^{-7}$ cm/s

Liquid head: $h = 30$

Suction potential at the liner bottom: $-h_d = -500$ cm.

Thus, $v = 3.65 \times 10^{-7}$ cm/s, and the parameter $D/vd = 0.0137$. From Figure 8-6, parameter $vt/\phi d = 0.7$ and the predicted transit time is 6 years.

8.4.3.6 Numerical Solution of the Flow Equation--

Since solute transport is not considered, the time required for solute chemical breakthrough cannot be determined. For highly unsaturated liners, the time required for the solute chemical breakthrough may be approximated by that required for the saturation front breakthrough and may be determined as above.

8.4.3.7 Numerical Solution of the Solute Transport Equation--

The computer program SOILINER, which was originally developed to solve the flow equation, was modified to include the advective and diffusive solute transport. Only molecular diffusion was considered to describe the dispersion process. This modification solves the flow and solute transport equations simultaneously. First the solution of the flow equation is advanced by a flow time step. The computed values of the fluxes and moisture contents are then used to advance the solution of the solute transport equation. For stability and accuracy, it was often necessary to subdivide the flow time steps into smaller solute transport time steps. The computations were carried out for the time period of 10 years for three initial liner moisture contents of 50 percent, 80 percent, and 95 percent of saturation. Solute breakthrough, as defined above, was not observed in any of the cases during the 10-year period. The depth of the penetration corresponding to $C/C_o = 0.1$ was noted for various intermediate times, and the time required for the solute breakthrough at the liner bottom was then extrapolated (see Figure 8-8). These extrapolated breakthrough times are as shown in Table 8-6.

As the results indicate, the transit time decreases as the initial liner moisture content decreases. This contrasts with the flow equation, where transit time (based on advancement of the saturation front) decreases as liner moisture content increases. As the initial liner moisture content increases, the time required for liner saturation decreases as indicated by the flow equation. However, the initial seepage is due to displacement of liner moisture, and, therefore, the transit time based on leachate chemicals is longer with greater initial liner moisture content. Also, it is interesting that for the case of 50 percent saturation the transit time based on both the flow and solute transport equations are close together. Therefore, it appears that where the initial saturation is 50 percent or lower, it may be sufficient to use the flow equation alone to predict the breakthrough times. But for higher saturations, use of the flow equation alone is likely to underpredict the transit time required for solute breakthrough.

8.5 SUMMARY AND CONCLUSIONS

Transit time and associated performance criteria may be defined in two ways: on the basis of the movement of moisture in the liner, or on the

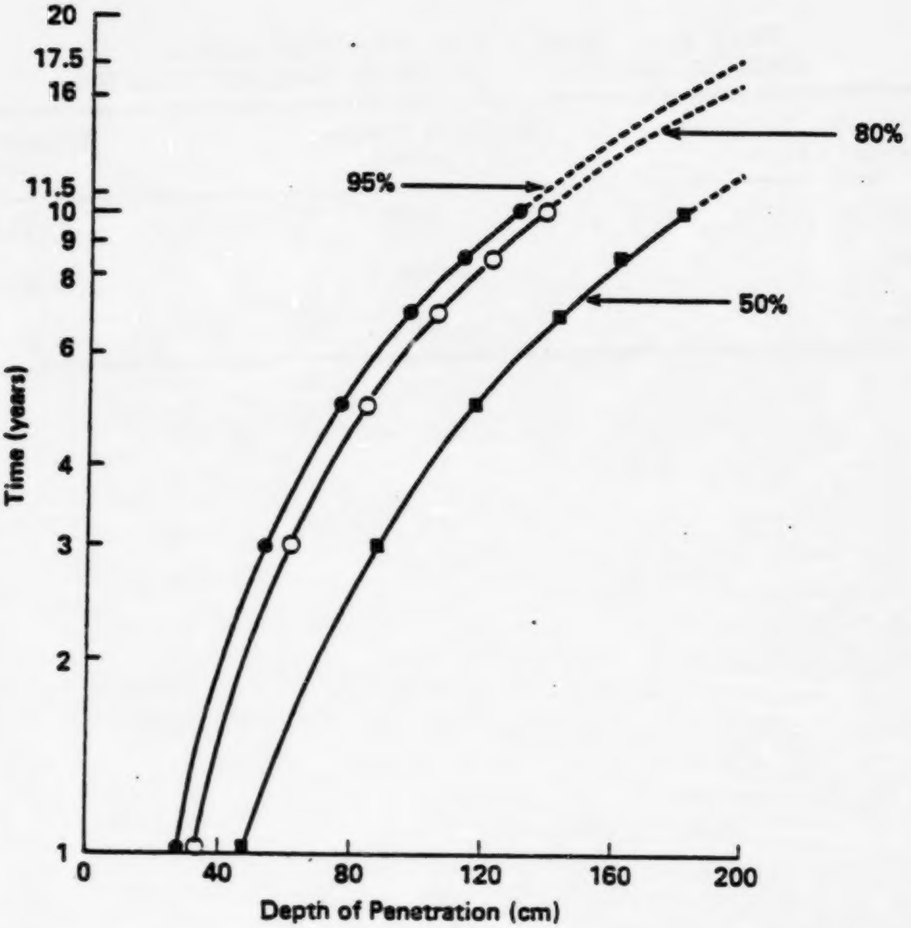


Figure 8-8. Extrapolation of transit times for solute transport, for initial liner saturations of 50%, 80% and 95%.

TABLE 8-6. TRANSIT TIME ESTIMATION USING A NUMERICAL SOLUTION TO THE SOLUTE TRANSPORT EQUATION

Initial liner saturation (%)	Moisture content by volume	Transit time (yr)
50	0.2475	11.5
80	0.396	16.0
95	0.47	17.5

basis of the movement of waste chemicals in the liner. Both of these definitions are recognized in the methods discussed. For criteria based on moisture movement, it is sufficient to consider an equation describing moisture flow in the liner (flow equation); for criteria based on chemical movement, it is also necessary to consider the equation(s) describing transport of the chemical(s) in the aqueous phase (solute transport equation).

The methods discussed in this section are arranged in ascending order of complexity. The simple transit time equation makes the most simplifying assumptions, requires the least amount of data, and is easy to use. All the analytical solutions are relatively easy to use compared to numerical methods because a single final equation is used for transit time prediction. Numerical solutions are expensive and time consuming but can also provide accurate solutions, with the accuracy dependent upon the quality of required data. Numerical solutions also require expertise in the available models and hence may not be easily accessible.

One of the most important elements for selecting a transit time prediction method is the initial degree of saturation of the liner. The simple and modified transit time equation and the transit time equation with diffusion assume a saturated liner and thus may be applicable where there is a high initial degree of saturation. On the other hand, the Green-Ampt model and the linearized infiltration equation are more applicable in situations where the initial degree of saturation in a liner is low. The performance criteria or the definition of "transit time" are also important in the method selection process. If the transit time is based on liner saturation, obviously the three transit time equations based on a saturated liner will not be applicable since they will indicate zero transit time. If the transit time is based on chemical transport, the Green-Ampt model and the linearized infiltration equation may not be applicable because they do not consider the solute transport equation. Numerical solutions of the flow and solute transport equations together will of course be applicable in all situations.

8.6 DEFINITION OF TERMS

This section provides an explanation of various terms frequently used in the discussion of transit time prediction methods. Where a symbol is associated with a term, it is listed. The abbreviations in parentheses stand for dimensions of the unit and are defined as follows:

D = dimensionless

L = length

T = time

M = mass.

Capillary action (capillarity). The rise or movement of fluid in the interstices of a soil due to capillary forces.

Capillary suction potential, Ψ (L). The potential expressed in head of water that causes the water to flow by capillary action. The word "suction" is used to emphasize the negative sign of this potential. This term is generally used to indicate pressure head of water in the unsaturated zone.

Chemical flux (M/L^2 T). Rate of flow of a chemical species expressed as mass per unit cross-sectional area per unit time. The chemical flux may be obtained by multiplying total volumetric flux by chemical concentration.

Concentration, C (M/L^3). Concentration of a chemical species in the aqueous phase expressed as mass per unit volume.

Darcian velocity, v (L/T). Velocity of aqueous phase based on total cross-sectional area obtained by dividing volumetric flow rate by cross-sectional area. The actual velocity of water in the soil is obtained by dividing the Darcian velocity by soil porosity.

Degree of saturation (%). Percent of the void volume in soil that is filled by water. Same as moisture content expressed in terms of percent.

Dispersion coefficient, D (L^2/T). Describes spreading of the dissolved species due to hydrodynamic or mechanical dispersion and molecular diffusion. The first process is caused by mixing due to variations in fluid velocities associated with distance from pore walls. Diffusion occurs in response to concentration gradients and by random thermal motion.

Flux (L/T or M/L^2T). Flow rate per unit area. The first group of dimensions apply when the flow rate is expressed as volumetric flow rate, whereas the second group of dimensions apply when the flow rate is expressed as mass flow rate.

Leachate. This term is used to emphasize the chemical species in an aqueous medium. Leachate may have several chemical species in varying concentrations in an aqueous medium. Leachate may also be generated by organic solvents; however, in this section we are primarily concerned with aqueous medium.

Leachate flux (M/L^2T). This is the same as chemical flux described above.

Moisture content (volumetric), θ (D). Volume fraction of water in soil. This is obtained by multiplying porosity with fractional degree of saturation.

Moisture content (% v/v). Percentage of soil volume occupied by moisture.

Porosity, ϕ (D). Ratio of volume of voids in a soil mass to the total volume of the soil mass. This is often referred to as total porosity.

Only interconnected voids contribute to the water flow and when the volume of such interconnected voids is used, this ratio gives effective porosity.

Potential, Ψ (L). Expressed in head of water. Potential is a driving force, and in this report it is always expressed relative to atmospheric pressure; thus, this term is similar to pressure head.

Pressure head, Ψ (L). Expressed in head of water. Water pressure head defined relative to atmospheric pressure is given by:

$$\psi = \frac{(P_w - P_a)}{\rho_w g}$$

where

P_w = water pressure

P_a = pressure in air phase taken as atmospheric pressure

ρ_w = density of water

g = gravitational acceleration.

From this definition it is clear that Ψ is positive in the saturated zone and negative in unsaturated zone. The negative pressure head in the unsaturated zone arises due to the capillary suction.

Relative hydraulic conductivity, k_r (D). The ratio of hydraulic conductivity of a given soil at a certain moisture content with the hydraulic conductivity of the same soil in saturated condition.

Retardation factor, R (D). Accounts for linear first-order interactions between a chemical species and soil (e.g., adsorption).

Saturated hydraulic conductivity, K_s (L/T). Hydraulic conductivity of a saturated soil with respect to water.

Saturated medium. A porous medium in which all voids are filled with fluid under pressure greater than atmospheric pressure.

Seepage. Movement of water (vertically downward). This term refers to the total liquid flow through a porous medium; for example, a clay liner.

Seepage flux, q (L/T). Volumetric flow rate of seepage in a porous medium per unit cross-sectional area. In a clay liner flow domain, the seepage flux has the same value as the Darcian velocity.

Suction potential, Ψ (L). Same as potential except that the word "suction" is used to emphasize the negative sign of the potential or pressure head. In the case of an unsaturated medium the suction is due to capillary forces, so in this case the suction potential is the same as capillary suction potential.

Unsaturated medium. A porous medium in which only a fraction of the void volume is filled with fluid under less than atmospheric pressure. The term "partially saturated medium" means the same as "unsaturated medium" except that the moisture content is greater than zero.

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APPENDIX A

TEST METHOD DESCRIPTIONS

APPENDIX A

TEST METHOD DESCRIPTIONS

The descriptions presented in Table A-1 are only intended to give the reader general information about the specific test methods. These method descriptions are taken directly from Geotechnical Quality Assurance of Construction of Disposal Facilities (Spigolon and Kelly, 1984). More detailed information can be obtained from the references noted for the various test methods. Data sheets for some of the test methods are provided as illustrative examples. These data sheets are merely examples and their inclusion here does not connote that the authors recommend that they specifically be used.

TABLE A-1. LIST OF TEST METHODS

Method	Number
Parameter Measured: Water Content	
Standard Oven-Dry	1
Standard Nuclear Moisture/Density Gage	2
Gas Burner	3
Alcohol Burning	4
Calcium Carbide (Speedy)	5
Microwave Oven	6
Infrared Oven	7
Parameter Measured: Unit Weight	
Standard Laboratory Volumetric	8
Standard Laboratory Displacement	9
Standard Field Sand-Cone	10
Standard Field Rubber Balloon	11
Standard Field Drive-Cylinder	12
Standard Nuclear Moisture/Density Gage	13
Parameter Measured: Specific Gravity	
Standard Laboratory	14
Parameter Measured: Grain-Size Distribution	
Standard Sieve Analysis (+200 Fraction)	15
Amount of Soil Finer than No. 200 Screen (Wash) Standard	16
Standard Laboratory Hydrometer (-200 Fraction)	17
Pipette Method for Silt and Clay Fraction	18
Decantation Method for Silt and Clay Fraction	19
Parameter Measured: Liquid Limit	
Standard Multipoint	20
Standard One Point	21
Parameter Measured: Plastic Limit	
Standard Laboratory	22
Parameter Measured: Cohesive Soil Consistency	
Standard Unconfined Compression	23
Field Expedient Unconfined Compression	24
Hand Penetrometer	25
Handheld Torvane	26

(continued)

TABLE A-1 (continued)

Method	Number
Parameter Measured: Water Content/Density/Compactive Effort	
25 Blow Standard Proctor Compaction	27
25 Blow Modified Proctor Compaction	28
Nonstandardized Proctor Compaction	29
Rapid, One Point Proctor Compaction	30
Rapid, Two Point Proctor Compaction	31
Hilf's Rapid	32
Ohio Highway Department Nest of Curves	33
Harvard Minature Compaction	34

Method No. 1

Parameter Measured: Water Content

Title of Test Method: Standard Oven-Dry

Principle of Test Method: This method determines the water content of a soil sample by first weighing it wet and then again after it has been dried in an oven.

Test Method

(1) Apparatus: Drying oven (thermostatically controlled, preferably of the force-draft type), balance sensitive to 0.01 g., specimen containers (tares) with lids, and a desiccator.

(2) Procedure: The procedure for this method consists simply of taking a specimen of known weight, placing it into an oven and drying it at a particular temperature and specified time. Upon drying the specimen is removed, reweighed and the moisture content is calculated.

(3) Reference: ASTM D 2216.

Limitations: With many soils, close control of water content during field compaction is necessary to develop a required density, strength and hydraulic conductivity in the soil mass. Oven-drying is the standard test for determining water content of soils in the geotechnical engineering practice. However, the method does not lend itself easily to field use. Although temperature controlled ovens are currently available on some construction sites, they require 4 to 12 hours for drying which may be excessive for the close control of field compaction. All soils can be tested for moisture content by oven-drying.

Status of the Method: Oven-drying of soil is the accepted laboratory method among the geotechnical engineering profession for determination of water content.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the wet weight and the dry weight. An example data sheet is provided.

Name:__________________________ Sample Numbers__________________________

Date:__________________________ Sheet Number __________________________

WATER CONTENT DETERMINATIONS
DATA AND COMPUTATION SHEET

NOTES: Tare is weight of container (watch glasses and clip, Petri dishes, can, etc.)

$$\text{Water Content} = w = \frac{\text{Wt. of Water}}{\text{Wt. of Dry Soil}} \cdot 100\%$$

Sample Number						
Type of Test						
Container Number						
Wt. Sample + Tare Wet						
Wt. Sample + Tare Dry						
Wt. of Water						
Tare						
Wt. of Dry Soil						
Water Content						

Sample Number						
Type of Test						
Container Number						
Wt. Sample + Tare Wet						
Wt. Sample + Tare Dry						
Wt. of Water						
Tare						
Wt. of Dry Soil						
Water Content						

Sample Number						
Type of Test						
Container Number						
Wt. Sample + Tare Wet						
Wt. Sample + Tare Dry						
Wt. of Water						
Tare						
Wt. of Dry Soil						
Water Content						

Method No. 2

Parameter Measured: Water Content

Title of Test Method: Standard Nuclear Moisture/Density Gage

Principle of Test Method: This method measures in place water content by directing fast neutrons of known intensity into the soil and measuring the intensity of slow or moderated neutrons reflected back.

Test Method

(1) Apparatus: Fast neutron source, slow neutron detector (readout device and housing) and reference standard and site preparation device.

(2) Procedure: This method allows the determination of water content of soil and soil aggregate in place through the use of nuclear equipment. The equipment is calibrated to determine water content, as weight of water per unit volume of material. Water content as normally used is defined as the ratio, expressed as a percentage, of the weight of water in a given soil mass to the weight of solid particles. It is determined with this procedure by dividing the water content by the dry unit weight of the soil. Therefore, computation of water content using the nuclear equipment also requires the determination of the dry unit weight of the material being tested. Most available nuclear equipment has the provision for measuring both the water content and the wet unit weight. The difference between these two measurements gives the dry unit weight.

(3) Reference: ASTM D 3017.

Limitations: The method described is useful as a rapid, nondestructive technique for the in-place determination of the water content of soil. The fundamental assumptions inherent in the method are that the hydrogen present is in the form of water as defined by ASTM D 2216, and that the material under test is homogeneous. Test results may be affected by chemical composition, sample heterogeneity, and, to a lesser degree, material density and the surface texture of the material being tested. The technique also exhibits spatial bias in that the apparatus is more sensitive to certain regions of the material under test. The nuclear method, which is applicable to a wide range of soils, requires operation by an experienced technician in order to obtain reliable measurements. A weakness in the nuclear method is that a sample is not taken to determine the water content, and thus the test results cannot be compared to the other water content methods, e.g. the oven-dry method. In addition, this method requires equipment that utilizes radioactive materials which themselves may be hazardous to the health of the operator. Effective operator instructions together with routine safety procedures are essential to the proper operation of this type of equipment.

Status of the Method: Nuclear gages offer a rapid and accurate means for obtaining water content values for a wide variety of soils. Recent advances in the design of nuclear equipment and a better understanding of the nuclear principles involved have led to increasingly widespread use of nuclear gages in earthwork construction control.

DATA SHEET FOR FIELD DENSITY TEST
NUCLEAR GAUGE METHOD

A	Moisture Standard Count		Gauge Type	Serial No
	Density Standard Count			

FOR

JOB	JOB NO.
SOURCE OF MATERIAL	DATE

	TEST LOCATION							
	ELEVATION							
	TEST NUMBER							
C	Moisture Count							
D	Moisture Count Ratio	$\frac{C}{A}$						
E	Density Count							
F	Air-Gap Count (if used)							
	Density Count Ratio	$\frac{E}{B}$ or $\frac{E}{F}$						
H	Density, Wet Wt., PCF	From G & Chart						
I	Moisture Content, PCF	From D & Chart						
J	Density, Dry Wt., PCF	H - I						
K	MOISTURE CONTENT, PERCENT	$\frac{I}{J}$						
L	OPTIMUM MOISTURE, PERCENT	See Curve						
M	DENSITY, DRY WT., PCF	From J						
N	THEORETICAL DENSITY, PCF	See Curve						
O	PERCENT COMPACTION	$\frac{M}{N}$ x 100						
P	REQUIRED PERCENT COMPAC.	See Specs.						
Q	MODE & PROBE DEPTH, IN.							
R	TYPE OF MATERIAL							

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Laboratory No.	Technician(s)

Calibration Procedure: The apparatus must be calibrated against a reliable direct method (e.g. oven-drying).

Documentation of Test: Items to be recorded include the water content and the wet unit weight. An example data sheet is provided.

Method No. 3

Parameter Measured: Water Content

Title of Test Method: Gas Burner

Principle of Test Method: This method determines the water content of a soil sample by first weighing it wet and then again after it has been dried.

Test Method

(1) Apparatus: Gas stove, frying pan, balance and stirring rods.

(2) Procedure: This method determines the approximate water content of soils by means of a gas-burner stove. The gas burner method involves the weighing of a moist sample, placing the sample in a pan on the stove and drying it to a constant weight, with occasional stirring of the sample to prevent burning. The dry sample, first permitted to cool, is then reweighed and the water content is determined.

(3) Reference: N/A

Limitations: The gas-burner method is used extensively for testing gravelly material. Two or more samples may be tested concurrently. When care is exercised to prevent overheating or burning of the sample, the testing time is usually about 1/2 hour. The method is inaccurate for organic soils or for those soils containing particles with loosely bound water, unless the drying time is accomplished at a temperature of not more than 140°F (60°C) for 1 hour or longer.

Status of the Method: The gas-burner method is used extensively as a rapid method for testing gravelly soils in the field control of earthwork.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the wet weight and the dry weight. No example data sheet is provided.

Method No. 4

Parameter Measured: Water Content

Title of Test Method: Alcohol Burning

Principle of Test Method: This method determines the water content of a soil sample by first weighing it wet and then again after it has been dried by alcohol burning.

Test Method

(1) Apparatus: Metal pan, balance, denatured alcohol and stirring rods.

(2) Procedure: This method determines the approximate water content of soil by burning alcohol that has been added to the soil. The general procedure consists of placing a weighted quantity of moist soil in a pan, adding alcohol to it and stirring the mixture, then igniting the alcohol. After ignition and the complete removal of the moisture by burning, the sample is reweighed and its water content is calculated.

(3) Reference: N/A

Limitations: The alcohol burning test is a rapid inexpensive method for determining the water content of soils. The method is usable with non-cohesive and cohesive soils. The method should not be used if the soil contains a large proportion of clay, gypsum, calcareous matter or organic matter. A large quantity of alcohol is required for testing coarse gravelly material. For multiple burnings, the testing time can be in excess of 1/2 hour. In terms of safety, the alcohol burning test possesses the potential for fire. Care should be exercised not to have alcohol on hand or in an open storage container near the testing apparatus during the ignition phase of the test. The alcohol should be stored in a safety container.

Status of the Method: The alcohol burning method of obtaining the water content of soil in the field has provided satisfactory results. Its use in the field has been well established.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the wet weight and the dry weight. No example data sheet is provided.

Method No. 6

Parameter Measured: Water Content

Title of Test Method: Microwave Oven

Principle of Test Method: This method determines the water content of a soil by first weighing it wet and then again after it has been dried in the oven.

Test Method

(1) Apparatus: Microwave oven suitable for drying, balance, specimen containers.

(2) Procedure: The procedure for this method is the same as that for the Standard Oven-Dry method. That is, a specimen is placed wet onto a balance and weighed. It is then placed in a microwave oven and dried completely. It is then weighed again. The weight difference was the water content.

(3) Reference: N/A

Limitations: Microwave ovens are not noted for their drying ability. There are necessary safety precautions when using a microwave oven.

Status of the Method: This method is not commonly used. Although a microwave oven heats much more rapidly than a conventional oven, it is an erratic dryer at best. Thus this method should probably only be used to determine water contents for soils which can be expected to have a low value, i.e. relatively low water content.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the wet weight and the dry weight. No example data sheet is provided.

Parameter measured: Water Content

Title of Test Method: Calcium Carbide (Speedy)

Principle of Test Method: This method determines the water content of a soil sample by measuring the pressure developed when measured quantities of the soil sample and powdered calcium carbide are mixed.

Test Method

(1) Apparatus: Calcium carbide pressure moisture tester, tared scale, two each 1-1/4 in. steel balls, brush, cloth and scoop.

(2) Procedure: The calcium carbide gas pressure method for determining water content consists of mixing measured quantities of moist soil and powdered calcium carbide in a closed chamber and measuring the pressure developed by the formation of acetylene gas. The reaction of calcium carbide and water forms acetylene gas and calcium hydroxide. The pressure developed is directly related to the amount of water entering into the reaction.

(3) Reference: AASHTO T 217.

Limitations: Only two sizes of testers are commercially available to test for water content using this method: (1) 26 gram capacity model and (2) a six-gram capacity model. The small chamber capacities of these devices control the soil sample size to be used. As a result, this method is unsuitable for representative samples of coarse granular material. AASHTO T 217 recommends that this method should not be used on granular material having particles large enough to affect the accuracy of the test (i.e. a soil with a grain-size distribution such that an appreciable amount would be retained on 4.75 mm sieves). If a six-gram sample is used (according to AASHTO T 217), the sample should not contain any particles that will be retained on the 2.00 mm sieve. The testing of heavy clays with this method requires special handling.

Status of the Method: The apparatus for this method is relatively inexpensive and well adapted to field testing. Use in the United States is extensive. In the field the calcium carbide method has been used extensively in the control of embankment construction. Normal testing time is less than 10 min. A moderate amount of operator training is required. The calcium carbide method is a standard method for rapid water content determination referenced under AASHTO.

Calibration Procedure: A calibration curve is required.

Documentation of Test: Items to be recorded include soil sample weight, powdered calcium carbide weight and the resultant gas pressure. No example data sheet is provided.

Method No. 7

Parameter Measured: Water Content

Title of Test Method: Infrared Oven

Principle of Test Method: This method determines the water content of a soil by first weighing it wet and then again after it has been dried in the oven.

Test Method

(1) Apparatus: Infrared oven suitable for drying, balance, specimen containers.

(2) Procedure: The procedure for this method is the same as that for the Standard Oven-Dry method. That is, a specimen is placed wet onto a balance and weighed. It is then placed in an infrared oven and dried completely. It is then weighed again. The weight difference was the water content.

(3) Reference: N/A

Limitations: There are necessary safety precautions when using an infrared oven. Infrared ovens are generally not widely available.

Status of the Method: This method is not commonly used. It can yield rapid results, however. The method should be desirable to a large operation where the benefits of rapid results outweigh the costs of the limitations above and the initial investment.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the wet weight and the dry weight. No example data sheet is provided.

Method No. 8

Parameter Measured: Unit Weight

Title of Test Method: Standard Laboratory Volumetric

Principle of Test Method: This method determines the unit weight of an undisturbed soil sample by measuring its weight and volume.

Test Method

(1) Apparatus: Sampling tools, balance, water bath, volume measuring device, oven and coating material (e.g. paraffin).

(2) Procedure: This method determines the density of cohesive soil in its natural state, compacted cohesive soil, and stabilized soil by measuring the weight and volume of undisturbed samples. The method briefly consists of cutting out a block of soil, coating it with a known amount of paraffin, weighing to obtain the net weight of the sample and immersing it in an overflow volumeter to determine the net volume of the sample, then dividing through for the unit weight.

(3) Reference: AASHTO T 233.

Limitations: The method is suitable for any material that remains intact during sampling. This metod is particularly adaptable to irregularly shaped specimens and soil containing gravel shells, etc. Sample size is not limited; large samples with coarse aggregate can be tested. This method is time consuming.

Status of the Method: This method is a nonstandard test when used in relation to compaction control. However, periodic record sampling on compacted embankments for dams usually entails obtaining block samples. In addition, if the sample is properly removed from the fill, this test provides for index and engineering properties.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the sample's net weight and net volume. An example data sheet is provided.

ORDER NO.________

CLIENT'S NO.________

DATE________

UNIT WEIGHTS

PROJECT________ LOCATION________ TESTED BY________

SAMPLE NO.						
ORIGINAL SAMPLE SIZE						
WT. CONTAINER + WET SOIL, GRAMS (W)						
WT. CONTAINER + DRY SOIL, GRAMS (W')						
WT. MOISTURE, GRAMS ($W_w = W - W'$)						
WT. CONTAINER + DRY SOIL, GRAMS (W')						
WT. CONTAINER, GRAMS (W'')						
WT. DRY SOIL, GRAMS ($W_s = W' - W''$)						
% MOISTURE, ($W = W_w/W_s \times 100$)						
DRY UNIT WT. LBS./FT.^{3}						

SAMPLE NO.						
ORIGINAL SAMPLE SIZE						
WT. CONTAINER + WET SOIL, GRAMS (W)						
WT. CONTAINER + DRY SOIL, GRAMS (W')						
WT. MOISTURE, GRAMS ($W_w = W - W'$)						
WT. CONTAINER + DRY SOIL, GRAMS (W')						
WT. CONTAINER, GRAMS (W'')						
WT. DRY SOIL, GRAMS ($W_s = W' - W''$)						
% MOISTURE, ($W = W_w/W_s \times 100$)						
DRY UNIT WT. LBS./FT.^{3}						

SAMPLE NO.						
ORIGINAL SAMPLE SIZE						
WT. CONTAINER + WET SOIL, GRAMS (W)						
WT. CONTAINER + DRY SOIL, GRAMS (W')						
WT. MOISTURE, GRAMS ($W_w = W - W'$)						
WT. CONTAINER + DRY SOIL, GRAMS (W')						
WT. CONTAINER, GRAMS (W'')						
WT. DRY SOIL, GRAMS ($W_s = W' - W''$)						
% MOISTURE, ($W = W_w/W_s \times 100$)						
DRY UNIT WT. LBS./FT.^{3}						

Method No. 9

Parameter Measured: Unit Weight

Title of Test Method: Standard Laboratory Displacement

Principle of Test Method: This method determines the unit weight of a soil sample by determining the weight of the sample and the volume of its hole.

Test Method

(1) Apparatus: Sampling tools, soil tray and pans, balances, measure, drying equipment, two gallons of lubricating oil, and a gauge point.

(2) Procedure: This method determines the density of soil in-place by finding the mass and water content of a disturbed sample and measuring the volume occupied by the sample using an oil of a known density. The method can be performed fairly fast, however a chief concern is the mess caused by the oil. The general procedure consists of leveling the test site, digging a hole in the compacted earthwork, weighing the material removed, measuring the volume of the hole by placing a measured quantity of oil in it and calculating the wet unit weight by dividing the weight of the moist soil by the volume of the hole.

(3) Reference: AASHTO T 214.

Limitations: This method may be used in testing materials with both fine and coarse particles; however, the test is best suited for soils and soil-aggregate mixtures that are relatively impervious. The method may not be suitable for testing materials having fissures, cracks, or large voids.

Status of the Method: The oil displacement method is a conventional test in the control of earthwork construction. However, it often provides less satisfactory results than the sand-cone method.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the weight of the soil sample and the volume of the oil. No example data sheet is provided.

Method No. 10

Parameter Measured: Unit Weight

Title of Test Method: Standard Field Sand-Cone

Principle of Test Method: This method determines unit weight by determining the weight of a soil sample and the volume of its hole.

Test Method

(1) Apparatus: One gallon jar, double cone assembly, base plate and accessories.

(2) Procedure: The test consists of digging out a sample of the material to be tested and weighing it. The volume of the hole is then determined by using the sand-cone.

(3) Reference: ASTM D 1556.

Limitations: The sand-cone has features which limit its usefulness. The method can be used satisfactorily, however, if these limitations are recognized and proper precautions observed. The poured density of sand is affected by atmospheric moisture and changes in relative humidity which means the sand should be calibrated before use. Care should be exercised to avoid jarring and densifying the sand during the filling procedure and the test should not be conducted during vibration of the site such as occurs during the use of heavy equipment. Sample size is limited by sand supply.

Status of the Method: The sand-cone test is a conventional test method for earthwork control. The method is reliable and the most commonly used test to determine the density of in-place soil. On Corps of Engineers earthwork projects the test serves as the referee test for all other control tests used. The sand-cone test is widely used in cohesive soils and can be also used in soils that are of low plasticity as well as gravelly soils. The test is not applicable in clean sands or gravel and loose granular material. The method is applicable to large and small projects.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the weight of the soil sample and the volume of the sand. No example data sheet is provided.

Method No. 11

Parameter Measured: Unit Weight

Title of Test Method: Standard Field Rubber Balloon

Principle of Test Method: This method determines unit weight by determining the weight of a soil sample and the volume of its hole.

Test Method

(1) Apparatus: Calibrated vessel, elastic membrane, pressure control device, baseplate and accessories.

(2) Procedure: This method determines the in-place density by removing soil from a hole, weighing the excavated material, and measuring the volume of the hole by a liquid-filled (water) balloon under constant pressure. Chief advantages of this method are its operation simplicity and speed with which tests can be conducted.

(3) Reference: ASTM D 2167.

Limitations: For general use in clays and consolidated sands, the rubber balloon apparatus provides good results. The method is not suitable for very soft soil which will deform under slight pressure or in which the volume of the hole cannot be maintained constant. The method is not well adapted to the measurement of volumes in loose granular material. However, of all in-place density tests some soil engineers recommend the water balloon as the preferred method for granular soils. The tests is well adapted to small and large projects. Physical limitation of the apparatus restricts the size of the test hole to approximately four or six inches in diameter and from six to twelve inches in depth.

Status of the Method: The water balloon test is widely used for determining in-place density for the control of earthwork. The method is used by the Corps of Engineers and other agencies because of its application to a wide range of materials and its past performance record.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the weight of the soil sample and the volume of water. No example data sheet is provided.

Method No. 12

Prameter Measured: Unit Weight

Title of Test Method: Standard Field Drive-Cylinder

Principle of Test Method: This method determines the unit weight of a soil sample by securing one in a known-volume tube and then weighing it.

Test Method

(1) Apparatus: Acceptable drive cylinder, drive head, straightedge, shovel, weight, scales, drying oven and airtight containers.

(2) Procedure: The drive-cylinder method for determining in-place density involves obtaining a relatively undisturbed soil sample by driving a thin-walled cylinder into the soil with a special driving head. Two procedures are described in ASTM D 2937 for performing this test, one for testing at the surface or at very shallow depths, usually less than 3 ft (1 m), and one for testing at greater depths. The general procedure for both depths consists of driving a sampling tube into the soil, withdrawing the tube with the sample, trimming the sample flush with the ends of the tube, weighing, then calculating the unit weight of the soil by dividing the net weight of the sample by the volume of the tube.

(3) Reference: ASTM D 2937.

Limitations: The drive-cylinder method of determining in-place density can be used satisfactorily in moist, cohesive, fine-grained soils and in many sands which exhibit tendencies toward cohesiveness. The method is not appropriate for sampling very hard soils which cannot be penetrated easily, or for soils of low plasticity which are not readily retained in the cylinder. The method sample size is limited by the sample tube. The chief disadvantage of the test is that it is limited to fine-grained soils.

Status of the Method: The standard field drive-cylinder method is a conventional test method in earthwork control. It is, however, less accurate than the sand cone or water balloon methods.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the soil sample's weight. No example data sheet is provided.

Method No. 13

Parameter Measured: Unit Weight

Title of Test Method: Standard Nuclear Moisture/Density Gage

Principle of Test Method: This method determines unit weight by a gamma source and a gamma detector.

Test Method

(1) Apparatus: A nuclear source emitting gamma rays, a gamma ray detector and a counter.

(2) Procedure: This method determines the density of soil and soil-aggregate in place through the use of nuclear equipment. In general, the total or wet density of the material under test is determined by placing a gamma source and a gamma detector either on, into, or adjacent to the material under test. These variations in test geometry are presented as the back-scatter, direct transmission, or air gap approaches. The intensity of radiation detected is dependent in part upon the density of the material under test. The radiation intensity reading is converted to measured wet density by a suitable calibration curve. Some commonly used sources of gamma rays are radium, cobalt 60 and cesium 137.

(3) Reference: ASTM D 2922.

Limitations: The method described is useful as a rapid, nondestructive technique for the in-place determination of the wet density of soils and soil-aggregates. The fundamental assumptions inherent in the method are that Compton scattering is the dominant interaction and that the material under test is homogeneous. Test results may be affected by chemical composition, sample heterogeneity, and the surface texture of the material being tested. The technique also exhibits spatial bias in that the apparatus is more sensitive to certain regions of the material under test. The nuclear method is applicable to a wide range of soil. The nuclear method requires a considerably experienced operator in order to obtain reliable measurements. A weakness in the nuclear method is that a sample is not taken to determine the water content, and thus the test results cannot be compared to another unit weight method. In addition this method requires equipment that utilizes radioactive materials which may be hazardous to the health of the operator. Effective operator instructions together with routine safety procedures are essential to the proper operation of this type of equipment.

Status of the Method: Nuclear gages offer a rapid and accurate means of obtaining density values for a wide variety of materials. Recent advances in the design of nuclear equipment and a better understanding of the nuclear principles involved have led to increasingly widespread use of nuclear gages in earthwork construction control.

Calibration Procedure: The apparatus must be calibrated against a reliable direct method (e.g. standard field sand-cone).

Documentation of Test: Items to be recorded include the counter's gamma ray counts. No example data sheet is provided.

Method No. 14

Parameter Measured: Specific Gravity

Title of Test Method: Standard Laboratory

Principle of Test Method: This method determines the specific gravity of a soil sample by the use of a pycnometer.

Test Method

(1) Apparatus: A pycnometer and a balance.

(2) Procedure: The procedure for this method is to procure a soil sample, weigh it with the balance, and then determines its specific gravity by the use of the pycnometer.

(3) ASTM D 854.

Limitations: N/A

Status of the Method: This is the standard method. It is widely used.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the sample weight and the pycnometer reading. An example data sheet is provided.

Name: ______________________ Sample Numbers ______________

Date: ______________________ Sheet Number ______________

SPECIFIC GRAVITY DETERMINATIONS

Sample Number ______________ Date: ______________

Pycnometer Bottle Number: ______________ Evap. Dish No. ______________

Wt. Bottle + Water + Sample $= W_s =$ ______ Wt. Sample + Dish Dry = ______

Temperature of Suspension $= T =$ ______ Weight of Dish = ______

Wt. Bottle + Water at temp. $T = W_w =$ ______ Dry Wt. of Soil W_o = ______

$$s = \frac{W_o}{W_o + W_w - W_s} = \rule{3cm}{0.4pt} = \rule{2cm}{0.4pt} = \rule{2cm}{0.4pt}$$

Remarks: ______________________________

Sample Number ______________ Date: ______________

Pycnometer Bottle Number: ______________ Evap. Dish No. ______________

Wt. Bottle + Water + Sample $= W_s =$ ______ Wt. Sample + Dish Dry = ______

Temperature of Suspension $= T =$ ______ Weight of Dish = ______

Wt. Bottle + Water at temp. $T = W_w =$ ______ Dry Wt. of Soil W_o = ______

$$s = \frac{W_o}{W_o + W_w - W_s} = \rule{3cm}{0.4pt} = \rule{2cm}{0.4pt} = \rule{2cm}{0.4pt}$$

Remarks: ______________________________

Sample Number ______________ Date: ______________

Pycnometer Bottle Number: ______________ Evap. Dish No. ______________

Wt. Bottle + Water + Sample $= W_s =$ ______ Wt. Sample + Dish Dry = ______

Temperature of Suspension $= T =$ ______ Weight of Dish = ______

Wt. Bottle + Water at temp. $T = W_w =$ ______ Dry Wt. of Soil W_o = ______

$$s = \frac{W_o}{W_o + W_w - W_s} = \rule{3cm}{0.4pt} = \rule{2cm}{0.4pt} = \rule{2cm}{0.4pt}$$

Remarks: ______________________________

Parameter Measured: Grain-Size Distribution

Title of Test Method: Standard Sieve Analysis (+200 Fraction)

Principle of Test Method: This method determines the +200 fraction of a soil sample by the use of a No. 200 sieve.

Test Method

(1) Apparatus: A No. 200 sieve, that is a sieve with 200 openings per square inch, and balance.

(2) Procedure: This is a method dependent test. A dried soil sample is weighed and poured onto a No. 200 sieve and shook. That amount of soil not passing through the sieve is the +200 fraction. The +200 fraction is then weighed on the balance.

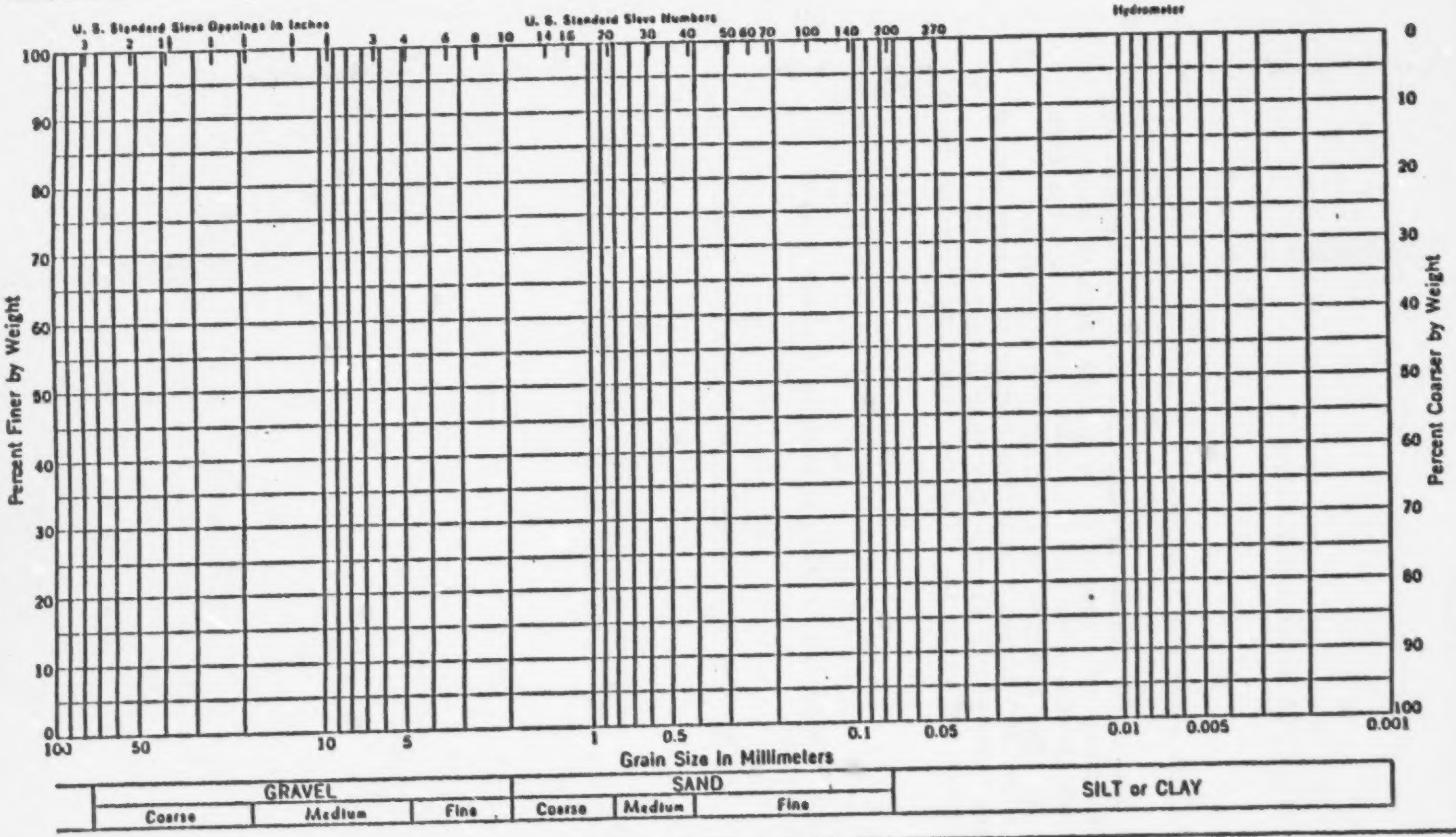
(3) Reference: ASTM D 422.

Limitations: The limitations of the method are its possible sources of error. These include: (a) overloading the sieve; (b) inadequate or incorrect shaking; and (c) broken or damaged sieves.

Status of the Method: This is the standard method. It is widely used.

Calibration Procedure: N/A

Documentation of Test: Items to be documented include the weight of the dried soil sample and the weight of the +200 fraction. An example data sheet is provided.



SAMPLE NO.	W.C.	LL.	PL.	PI	CLASSIFICATION

DRAWN	APPROVED	DATE	JOB No.

Parameter Measured: Grain-Size Distribution

Title of Test Method: Amount of Soil Finer than No. 200 Screen (Wash) Standard

Principle of Test Method: This method determines the -200 fraction of a soil sample by the use of a No. 200 sieve and washing.

Test Method

(1) Apparatus: A No. 200 sieve and a balance.

(2) Procedure: This method is similar to that described for determining the +200 fraction. However if the soil sample to be tested contains plastic fines, drying will cause them to adhere to the fine sand grains, and the test will yield erroneous results. The solution is to weigh the dry sample beforehand. Pour it on the No. 200 screen. Wash the sample to loosen the fines and allow them to pass through the sieve. Dry that amount of soil remaining on the No. 200 screen and weigh it. The two weight differences is then the fines content.

(3) Reference: ASTM D 422.

Limitations: The significant limitation of this method is the extra time required for washing and redrying.

Status of the Method: This is the standard method. It is widely used.

Calibration Procedure: N/A

Documentation of Test: Items to be documented include the dry weight of the sample and the dry weight of the +200 fraction. No example data sheet is provided.

Method No. 17

Parameter Measured: Grain-Size Distribution

Title of Test Method: Standard Laboratory Hydrometer (-200 Fraction)

Principle of Test Method: This method determines the grain-size distribution of the -200 fraction of a soil sample by the use of Stokes' Law and a standardized hydrometer.

Test Method

(1) Apparatus: A No. 200 sieve, a standardized hydrometer, a sedimentation cylinder, a thermometer and a breaker.

(2) Procedure: This method determines the grain-size distribution of that fraction of a soil sample passing the No. 200 sieve (the -200 fraction). The method utilizes a deflocculating (dispersing) agent and Stokes' Law to enable the different particle sizes to settle at different rates, thus enabling the technician to determine their distribution.

(3) Reference: ASTM D 422.

Limitations: An experienced technician is required to perform this test method. Considerable time is required for sample preparation. The test itself requires several hours to perform.

Status of the Method: This is the standard method. As it is the only "exact" method for determining the percent silt sizes and the percent clay sizes, it is widely used.

Calibration Procedure: N/A

Documentation of Test: Items to be documented include the dry weight of the sample and the dry weights of the settled fractions. Example data sheets are provided.

Name:________________________________ Sample and Test Number____________________

Date:________________________________ Sheet Number____________________

HYDROMETER ANALYSIS
DATA AND COMPUTATION SHEET

Evaporating Dish No. ____________________

Wt. Sample + Dish Dry = ____________________

Wt. Dish = ____________________

Dry Wt. W_o = ____________________

Hydrometer Number____________________

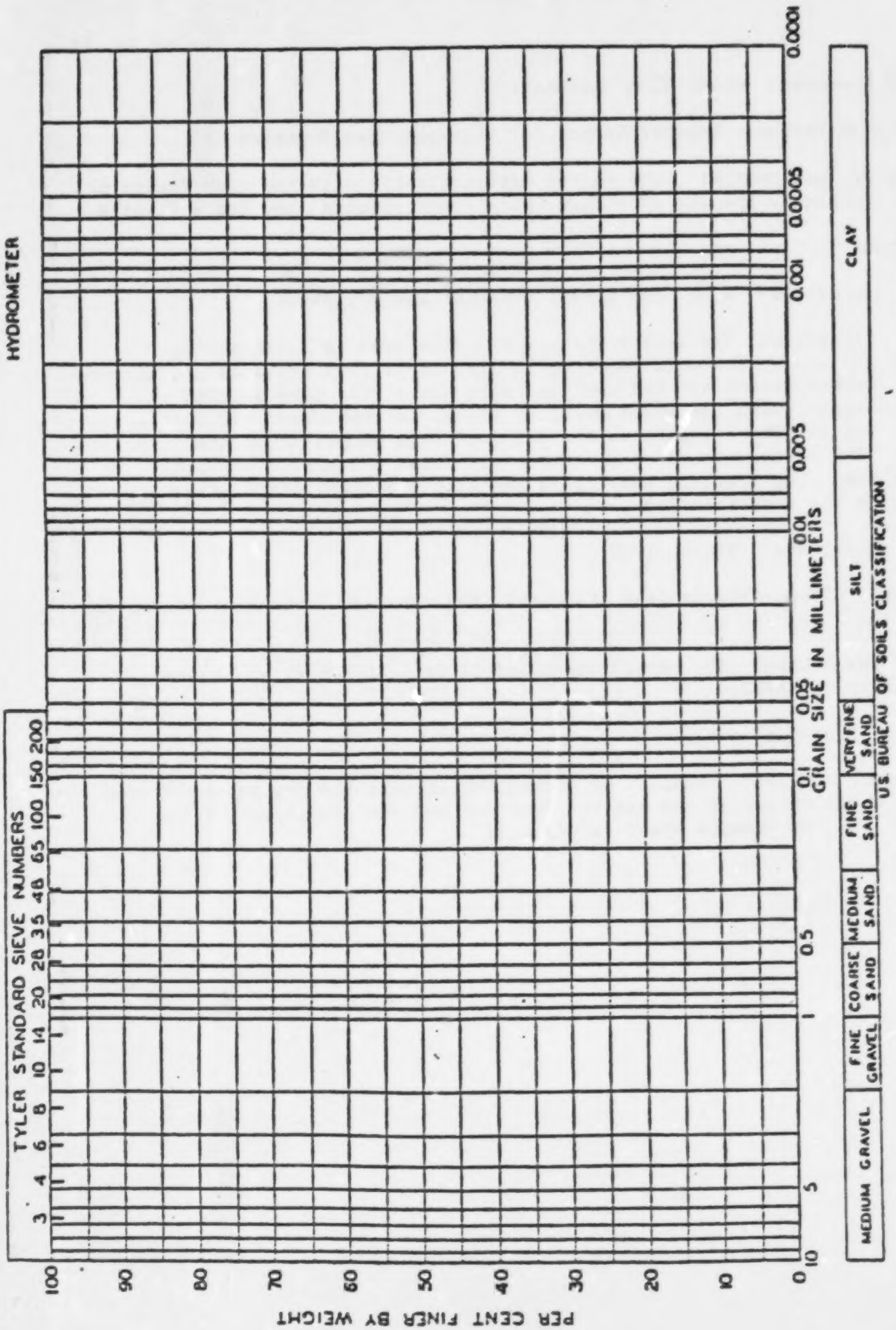
$$W\% = \frac{100}{W_o} \frac{s}{s-1} (R + m)$$

$W\% = \frac{100}{\_\_\_\_} \_\_\_\_ (R + m)$

$W\% = \_\_\_\_\_\_\_\_ (R + m)$

Meniscus Correction = c = ____________________

Date	Time	Temp.	Elapsed Time	R'	R=R'+c	Grain Diam	R + m	W%	Remarks



PROJECT______ BORING NO ______ SAMPLE NO ______

DEPTH______ ELEVATION______ REMARKS______

GRAIN SIZE DISTRIBUTION DIAGRAM

Method No. 18

Parameter Measured: Grain-Size Distribution

Title of Test Method: Pipette Method for Silt and Clay Fraction

Principle of Test Method: This method determines the silt and clay fractions of a soil sample by the use of a dispersing agent, Stokes' Law and a Pipette.

Test Method

(1) Apparatus: A No. 200 sieve, a beaker and a pipette.

(2) Procedure: The soil sample is deflocculated in a dispersing agent for one hour. The soil-water-agent mixture is then agitated and allowed to sit. Theroretically all the sand and silt sizes will have settled by then. The water, with the suspended clay sizes, is then drawn off by the pipette. The settled fraction is the original soil sample minus the clay fraction. The settled fraction is completely dried and passed through a No. 200 sieve. The fraction passing is the silt fraction, the fraction remaining is the sand fraction.

(3) Reference: Mills, 1970.

Limitations: The method is less exact than the standard laboratory hydrometer method.

Status of the Method: The method lends itself well to field laboratory use and is widely accepted.

Calibration Procedure: N/A

Documentation of Test: Items to be documented include the dry weight of the sample, the dry weight of the settled fraction and the dry weight of the silt fraction. No example sheet is provided.

Method No. 19

Parameter Measured: Grain-Size Distribution

Title of Test Method: Decantation Method for Silt and Clay Fraction

Principle of Test Method: This method determines the silt and clay fractions of a soil sample by the use of a dispersing agent, Stokes' Law and decantation.

Test Method

(1) Apparatus: A No. 200 sieve and a beaker.

(2) Procedure: The soil sample is deflocculated in a dispersing agent for one hour. The soil-water-agent mixture is then agitated and allowed to sit. Theoretically all the sand and silt sizes will have settled by then. The water, with the suspended clay sizes, is then drawn off by decantation (careful pouring). The settled fraction is the original sample minus the clay fraction. The settled fraction is completely dried and passed through a No. 200 sieve yielding the silt fraction.

(3) Reference: Mills, 1970.

Limitations: The method is less exact than the standard laboratory hydrometer method. As is also more rudimentary than the pipette method, it can be less accurate than this method in execution.

Status of the Method: The method lends itself well to field laboratory use and is widely accepted.

Calibration Procedure: N/A

Documentation of Test: Items to be documented include the dry weights of the sample, the settled fraction and the silt fraction. No example sheet is provided.

Method No. 20

Parameter Measured: Liquid Limit

Title of Test Method: Standard Multipoint

Principle of Test Method: This method determines the liquid limit of a soil sample by the use of the liquid limit device and a minimum of three trials.

Test Method

(1) Apparatus: Evaporating dish, spatula, liquid limit device, grooving tool and balance.

(2) Procedure: A soil sample is oven dried. Distilled water is added and mixed thoroughly with the sample till it is ready to be tested. A sample is placed in the liquid limit device and divided by the grooving tool. The test is run till the soil halves meet. The result is plotted. A minimum of three trials are performed.

(3) Reference: ASTM D 423.

Limitations: The method requires considerable time and a laboratory environment to be performed.

Status of the Method: For the purposes needed at a hazardous waste disposal facility the standard one point method will yield reasonable data with less effort and hence is more widely used.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the oven-dry weight of the sample, the water content and the number of blows. An example data sheet is provided.

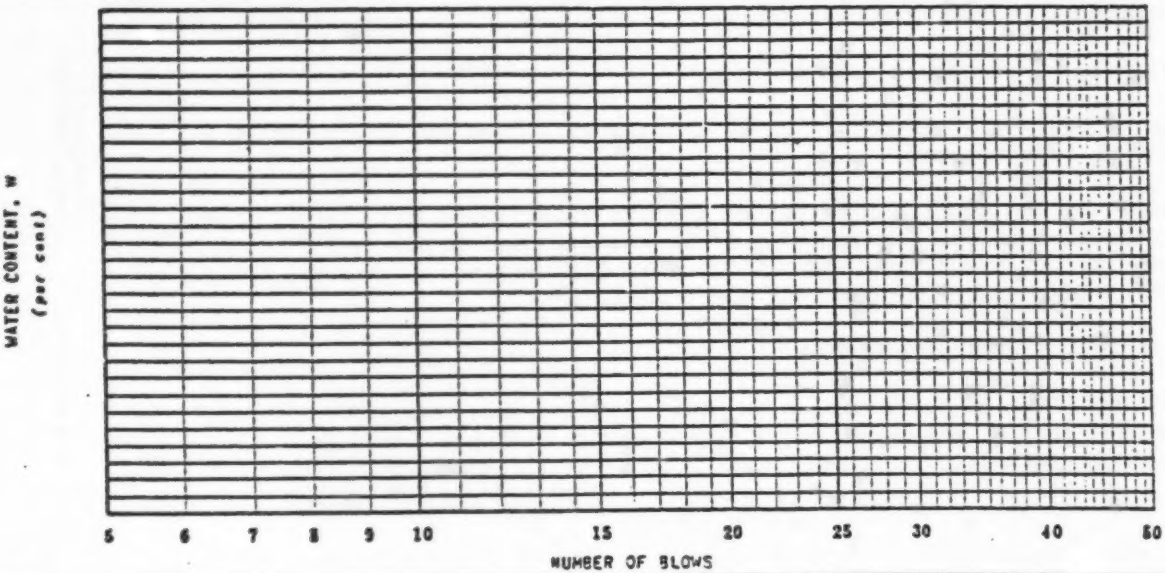
ATTERBERG LIMITS DETERMINATION	DATE

PROJECT	EXCAVATION NUMBER	SAMPLE NUMBER

LIQUID LIMIT, w_l

RUN NUMBER						
TARE NUMBER						
A. WEIGHT OF WET SOIL + TARE						
B. WEIGHT OF DRY SOIL + TARE						
C. WEIGHT OF WATER, W_w (A.-B.)						
D. WEIGHT OF TARE						
E. WEIGHT OF DRY SOIL, W_s (B.-D.)						
WATER CONTENT, $w = (\frac{W_w}{W_s} \times 100)$						
NUMBER OF BLOWS						

w_L	w_P	I_P ($w_L - w_P$)



PLASTIC LIMIT, w_P					NATURAL WATER CONTENT
RUN NUMBER					
TARE NUMBER					
F. WEIGHT OF WET SOIL + TARE					
G. WEIGHT OF DRY SOIL + TARE					
H. WEIGHT OF WATER, W_w (F.-G.)					
I. WEIGHT OF TARE					
J. WEIGHT OF DRY SOIL, W_s (G.-I.)					
WATER CONTENT, $w = (\frac{W_w}{W_s} \times 100)$					
PLASTIC LIMIT, I_P (Average w)					

REMARKS

TECHNICIAN (Signature)	COMPUTED BY (Signature)	CHECKED BY (Signature)

DD FORM 1 AUG 57 1209

GPO-931499

Method No. 21

Parameter Measured: Liquid Limit

Title of Test Method: Standard One Point

Principle of Test Method: This method determines the liquid limit of a soil sample by the use of the liquid limit device and one trial.

Test Method

(1) Apparatus: Evaporating dish, spatula, liquid limit device, grooving tool and balance.

(2) Procedure: A soil sample is oven dried. Distilled water is added and mixed thoroughly with the sample till it is ready to be tested. A sample is placed in the liquid limit device and divided by the grooving tool. The test trial is then run till the soil halves meet. The data is then plotted.

(3) Reference: ASTM D 423.

Limitations: The method requires considerable time and a laboratory environment to be performed.

Status of the Method: The method is more widely used than the standard multipoint method, but as it is a correlation test method and it is usually used in soil identification, it is often less practical than the commonly used visual-manual procedure (ASTM D 2488).

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the oven-dry weight of the sample, the water content and the number of blows. No example data sheet is provided.

Parameter Measured: Plastic Limit

Title of Test Method: Standard Laboratory

Principle of Test Method: This method determines the plastic limit of a soil sample, that is the lowest water content at which the soil can be rolled into 1/8 in. threads without breaking.

Test Method

(1) Apparatus: Evaporating dish, spatula, suitable containers, and balance.

(2) Procedure: For a given soil sample begin at a water content estimated to be greater than that at the plastic limit. A good start point would be the approximate liquid limit. Shape the soil into an ellipsoidal mass. Roll the sample into 1/8 in. threads. Cut the threads into 6 to 8 pieces. Repeat the process till the threads break at 1/8 in. An oven-dry moisture content determination at that point will yield the plastic limit.

(3) Reference: ASTM D 424.

Limitations: The method is simple and straightforward.

Status of the Method: This is the standard method and is widely used.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the oven-dry and wet weight of the sample at the plastic limit. An example data sheet is provided.

ATTERBERG LIMITS

SAMPLE Nº ____________ PROJECT ______________________ DATE ______________

SOURCE OF SAMPLE ____

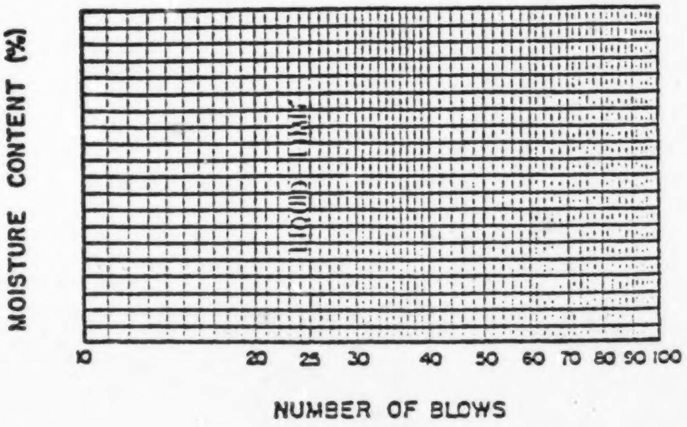
SUMMARY

NATURAL WATER CONTENT	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SHRINKAGE LIMIT

PLASTIC LIMIT				NATURAL WATER CONTENT		
DETERMINATION Nº	1	2	3	1	2	3
CONTAINER Nº						
CONTAINER + WET SOIL						
CONTAINER + DRY SOIL						
WEIGHT OF WATER						
CONTAINER + DRY SOIL						
WEIGHT OF CONTAINER						
WEIGHT OF DRY SOIL						
PERCENT WATER						

LIQUID LIMIT

DETERMINATION Nº	1	2	3	4	5	6
NUMBER OF BLOWS						
CONTAINER Nº						
CONTAINER + WET SOIL						
CONTAINER + DRY SOIL						
WEIGHT OF WATER						
CONTAINER + DRY SOIL						
WEIGHT OF CONTAINER						
WEIGHT OF DRY SOIL						
PERCENT WATER						



TESTED BY ____

Method No. 23

Parameter Measured: Cohesive Soil Consistency

Title of Test Method: Standard Unconfined Compression

Principle of Test Method: This method uses a compression device to determine the unconfined compressive strength of a soil sample: that is the load per unit area at which the specimen fails in simple compression.

Test Method

(1) Apparatus: Compression device, sample ejector, deformation indicator, vernier caliper, timer, balance and oven.

(2) Procedure: An unidsturbed sample is secured and a specimen prepared as per the reference. An estimate of the failure strength is made based on experience with a similar material. The specimen is placed in compression by uniformly progressive loads at 30 second intervals till failure or 20% strain is reached. The results are plotted.

(3) Reference: ASTM D 2166.

Limitations: The method requires a laboratory environment, the securing of an undisturbed sample (as described by ASTM Method D 1587), considerable specimen preparation, calculations and considerable time.

Status of the Method: The method is the standard means for unconfined compressive strength determination. However because of its limitations it is not commonly used in process control (because of more efficient methods) but is widely used in acceptance testing.

Calibration Procedure: The deformation indicator must be zeroed at the beginning of the test.

Documentation of Test: Items to be recorded include the water content of the sample as well as the strain deformations at their respective loads. An example data sheet is provided.

UNCONFINED COMPRESSION TEST - DATA SHEET

Soil Description ______________________ Project ______________________

______________________ Laboratory ______________________

Boring No. ________ Surface Elev. ________ Sample No. ________ Sample Depth Interval ________

LL ________ % w ________ % PL ________ % Date Tested ________

Pocket Penetrometer, P.P. ________ tsf Specific Gravity, G_s ________ Dry Unit Weight, γ_d ________ pcf

Soil Specimen Measurements

Diameter: ________ in. Initial Area, A_o: ________ sf Initial Length, L_o: ________ in

Proving Ring Conversion Factor: ________ Corrected Area, $A_c = \frac{A_o}{1 - \epsilon}$

WATER CONTENT

Specimen Location	Top	Middle	Bottom	Entire Remolded Sample
Container No.				
Wt. Cont. + Wet Soil(gms.)				
Wt. Cont. + Dry Soil(gms.)				
Wt. Container (gms.)				
Wt. Dry Soil(gms.)				
Wt. Water(gms.)				
Water Content(%)				

COMPRESSION TEST

Elapsed Time (Min.)	Load Dial Reading	Load (lbs.)	Vert. Dial (inches)	Axial Strain ϵ (%)	Corr. Area, A_c (sf)	Comp. Stress, S_c (psf)

Method No. 24

Parameter Measured: Cohesive Soil Consistency

Title of Test Method: Field Expedient Unconfined Compression

Principle of Test Method: This method uses a field expedient compression device to determine the unconfined compressive strength of a soil sample.

Test Method

(1) Apparatus: Sample selector, compression device, stress and deformation indicators, balance, oven and trimmer.

(2) Procedure: A sample is secured and prepared. An estimate of the sample's failure strength is made based on experience with a similar material. The specimen is then placed in compression by uniformly progressive loads till failure or a predetermined percent strain is achieved. The results are plotted.

(3) Reference: TM 5-530, 1971.

Limitations: The method involves a procedure virtually identical to that called for in the standard laboratory method. However the apparatus used in the field expedient method is less accurate, less care is given to sample preparation and the method itself is less accurate.

Status of the Method: The method lies between the standard laboratory method and the hand device methods in terms of accuracy and resources required. As the standard laboratory method is the accepted method for laboratory needs and the hand devices are sufficient for field testing. This method is not widely used.

Calibration Procedure: The stress and strain indicators must be zeroed at the beginning of the test.

Documentation of Test: Items to be recorded include the sample's water content as well as the stress and strain readings. No example data sheet is provided.

Parameter Measured: Cohesive Soil Consistency

Title of Test Method: Hand Penetrometer

Principle of Test Method: This method uses the hand penetrometer to determine the unconfined compressive strength of a soil sample.

Test Method

(1) Apparatus: Hand penetrometer with stress and strain indicators, trimming knife.

(2) Procedure: A sample is taken and trimmed by the hand penetrometer and a trimming knife. The test is then run with the device. One man can run the device and read the various stress and strain indicators. These values can be plotted and the unconfined compressive strength determined.

(3) Reference: Hvorslev, 1943.

Limitations: The sampling operation may cause a slight disturbance and decrease in strength of very brittle soils, a small downward deflection of soft soils, and a slight compaction of loose and partially saturated soils.

Status of the Method: The determination of cohesive soil consistency classification by this method is widely used as it is more accurate than the visual-manual method, and although it is less accurate than the standard laboratory method it is quite sufficient given the broad classifications for cohesive soil consistency.

Calibration Procedure: The stress and strain indicators must be zeroed at the beginning of the test.

Documentation of Test: Items to be recorded include the stress and strain readings during testing. An example data sheet is provided.

UNCONFINED COMPRESSION TEST

Soil Description ______________________ **Project** ______________________

______________________ **Laboratory** ______________________

Boring No. ________ **Surface Elev.** ________ **Sample No.** ________ **Sample Depth Interval** ________

LL ________ % **w** ________ % **PL** ________ % **Date Tested** ________

Pocket Penetrometer, P.P. ________ tsf **Specific Gravity,** G_s ________ **Dry Unit Weight,** γ_d ________ pcf

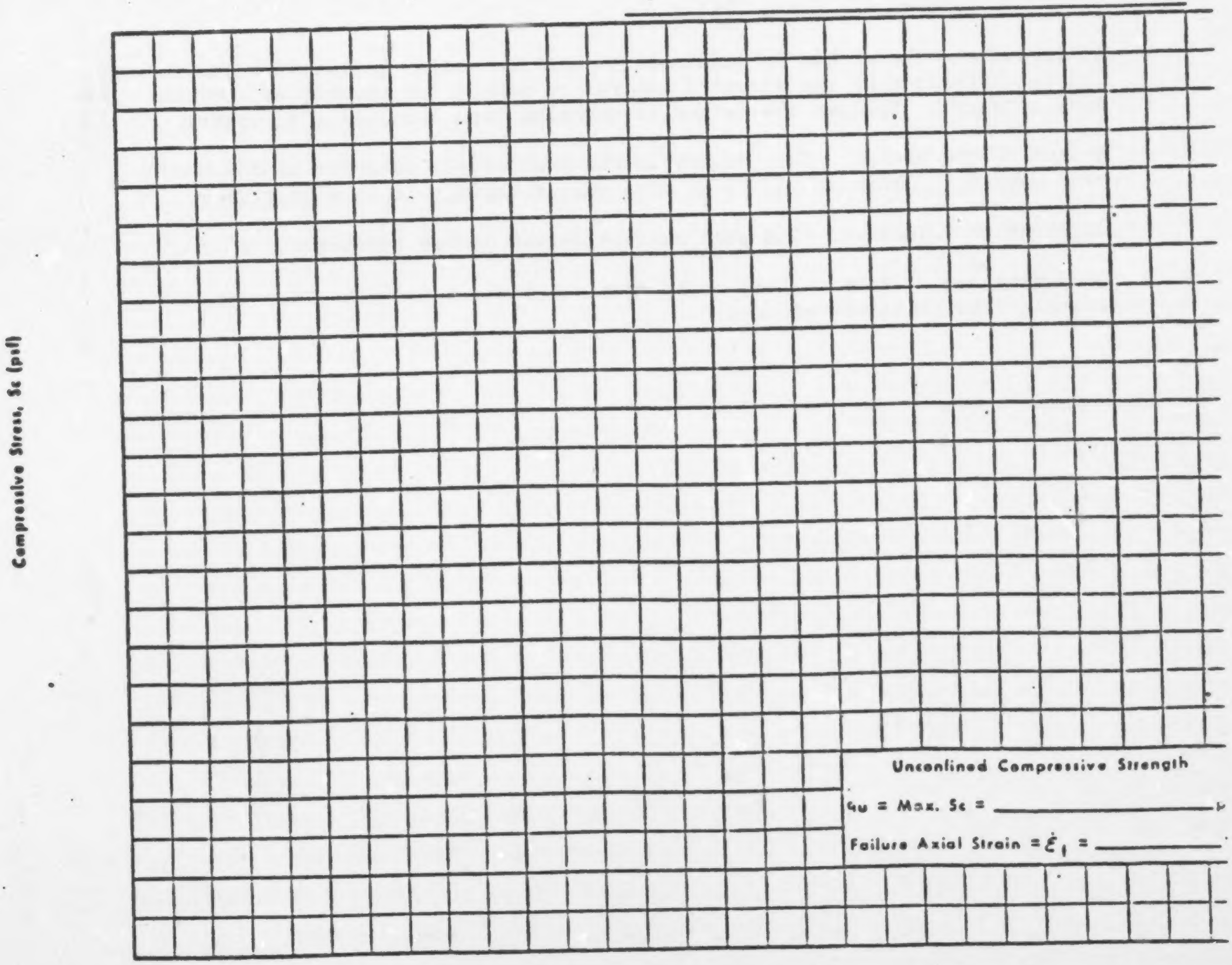
Axial Strain Rate: ________ inch/min.

Diagram of Failure:

Grain Size Distribution				
%Gr.	%C.S.	%F.S	% Silt	% Clay

Remarks: ______________________

____



Unconfined Compressive Strength

q_u = Max. Sc = ________ p

Failure Axial Strain = ε_f = ________

Method No. 26

Parameter Measured: Cohesive Soil Consistency

Title of Test Method: Handheld Torvane

Principle of Test Method: This method uses a handheld torvane to determine the soil shear strength of the in situ soil.

Test Method

(1) Apparatus: Handheld torvane with gage.

(2) Procedure: The handheld torvane is pushed through the soil crust to the desired depth. A rotary motion is then applied to the handle. The gage values are read and the soil shear strength, values for cohesion and the internal angle of friction, and consistency classification can be determined.

(3) Reference: Lanz, 1968.

Limitations: The method is standardized and the values gained from the method can be correlated to the standard laboratory method for unconfined compressive strength. However the method is somewhat less accurate than others.

Status of the Method: The torvane is the most widely accepted of the shear vane devices. Since it yields in situ values rapidly it is widely used.

Calibration Procedure: The gage must be zeroed before testing.

Documentation of Test: Items to be recorded include the gage readings. No example data sheet is included.

Parameter Measured: Water Content/Density/Compactive Effort

Title of Test Method: 25 Blow Standard Proctor Compaction

Principle of Test Method: This method determines the optimum moisture content and the maximum density of a soil sample by an impact compaction test. This is the original Proctor test.

Test Method

(1) Apparatus: Molds, rammer, extruder, balance, drying oven, sieves.

(2) Procedure: The specimen is prepared and placed in a mold. Most commonly the mold is the 4 inch mold. The soil is placed in three like layers. Each layer is compacted. Most commonly the compaction is 25 blows with a 5.5 pound hammer dropping 12 inches. Following compaction of all three layers the specimen is removed from the mold and trimmed. The mass of the sample is determined. This is divided by the volume of the mold and then the wet density of the sample, in pcf, is determined. The water content of the sample is then determined by oven drying. This entire procedure is repeated for at least four specimens. The four specimens' water contents should vary by about 1-1/2% between each sequential sample. The specimen water contents should bracket the estimated optimum moisture content. The values are plotted along with the "zero air voids curve." The optimum moisture content and maximum density of the sample can then be determined.

(3) Reference: ASTM D 698.

Limitations: This method requires a laboratory environment, the considerable time involved in sample drying and several hours to perform the test itself.

Status of the Method: This is the standard method. It is widely used. It is suited for the lesser compactive effort as might be applied to trench backfill. It may not be as suitable as the modified method for higher compactive efforts such as that which might be applied to a trench floor.

Calibration Procedure: The rammer must be calibrated before initial use and again after each 1,000 molds.

Documentation of Test: Items to be recorded include the mold volumes, specimen masses, and specimen water contents. An example data sheet is provided.

SOILS LABORATORY COMPACTION TESTS PROCTOR METHOD

Sample No.

SITE
HOLE
AREA
DEPTH

NUMBER OF BLOWS
PLASTIC LIMIT
SPECIFIC GRAVITY
DATE

RUN NO.	WEIGHT OF SAMPLE + CYLINDER KG.	WEIGHT OF CYLINDER KG.	WEIGHT OF SAMPLE KG.	WEIGHT OF SAMPLE LBS./CU. FT.	WATER ADDED GRAMS	WATCH GLASS NO.	WATER CONTENT % DRY WT.	REMARKS
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								

TESTED BY: DATE

COMPUTED BY: DATE

CYLINDER NO.

VOL. OF CYLINDER CU. FT.

UNIT WEIGHT FACTOR

CHECKED BY:

DATE

Parameter Measured: Water Content/Density/Compactive Effort

Title of Test Method: 25 Blow Modified Proctor Compaction

Principle of Test Method: This method determines the optimum moisture content and the maximum density of a soil sample by an impact compaction test.

Test Method

(1) Apparatus: Molds, rammer, extruder, balance, drying oven, sieves.

(2) Procedure: The specimen is prepared and placed in a mold in three like layers. The mold most commonly used is the four inch mold. Each layer is compacted. The compaction used for the four inch mold is 25 blows with a 10 pound hammer dropping 18 inches. Following compaction of all three layers the specimen is removed from the mold and trimmed. The mass of the sample is determined. This is divided by the volume of the mold and then the wet density of the sample, in pcf, is determined. The water content of the sample is then determined by oven drying. The entire procedure is repeated for at least four specimens. The four specimens' water contents should vary sequentially by about 1-1/2 percent. The specimen water contents should bracket the estimated optimum moisture content. The values are plotted along with the "zero air voids curve." The optimum moisture content and maximum density of the sample can then be determined.

(3) Reference: ASTM D 1557.

Limitations: This method requires a laboratory environment, the considerable time involved in sample drying and several hours to perform the test itself.

Status of the Method: This is also a standard method. It is widely used. As it better represents greater compactive effort than does the original Proctor test, it is more suitable for process control testing for trench floors and the like.

Calibration Procedure: The rammer must be calibrated before initial use and again after each 1,000 molds.

Documentation of Test: Items to be recorded include the mold volumes, specimen masses, and specimen water contents. No example data sheet is provided.

Method No. 29

Parameter Measured: Water Content/Density/Compactive Effort

Title of Test Methods: Nonstandardized Proctor Compaction

Principle of Test Methods: These methods determine the optimum moisture content and the maximum density of a soil sample by kneading compaction, static compression or other devices.

Test Methods

(1) Apparatus: Molds, sieves, balance, drying oven, extruder, kneading device or static compression device.

(2) Procedure: The procedures are method specific. (See the references.) However the procedures are very similar to the standard laboratory methods. Only the nature of the compactive effort and the specifics of the mold size, etc., differ.

(3) Reference: Johnson and Sallberg, 1962; and Antrim, 1970.

Limitations: These methods require a laboratory environment, the considerable time involved in sample drying and several hours to perform the tests themselves.

Status of the Method: These are not standard laboratory methods. They were developed for use in special situations. They are used by several state highway departments. They are not widely used in the laboratory because of the preference for the standard methods or in the field because of the ease and sufficiency of other methods.

Calibration Procedure: See the references.

Documentation of Test: Items to be recorded include the mold volumes, specimen masses, and specimen water contents. No example data sheet is provided.

Parameter Measured: Water Content/Density/Compactive Effort

Title of Test Method: Rapid, One Point Proctor Compaction

Principle of Test Method: This method determines the optimum moisture content and the maximum density of a soil sample by an impact compaction test and estimation, given one specimen test and its derived value.

Test Method

(1) Apparatus: Molds, rammer, extruder, balance, drying oven, sieves.

(2) Procedure: The procedure is essentially the same as that for the standard laboratory method. However, rather than running four or more (usually the number is five) specimens through the test and deriving a like number of points, only one test is run and one point is derived. The one point is at a water content estimated to be on the dry side of the optimum. From this point, and the well documented data on the soil, the optimum moisture content and maximum density can be determined.

(3) Reference: EM 1110-2-1911, 1977.

Limitations: This method depends upon good and well documented data on the local soils. The method also depends upon data which defines a relatively good line of optimums.

Status of the Method: For suitable soils this method provides a much more rapid means of process control. It is widely used. It is probable that this method could be sufficiently applied to the compaction of trench backfill.

Calibration Procedure: The rammer must be calibrated before initial use and again after each 1,000 molds.

Documentation of Test: Items to be recorded include the mold volumes, specimen masses, and specimen water contents. No example data sheet is provided.

Method No. 31

Parameter Measured: Water Content/Density/Compactive Effort

Title of Test Method: Rapid, Two Point Proctor Compaction

Principle of Test Method: This method determines the optimum moisture content and the maximum density of a soil sample by an impact compaction test and estimation, given two specimen tests and their derived values.

Test Method

(1) Apparatus: Molds, rammer, extruder, balance, drying oven, sieves.

(2) Procedure: The procedure is essentially the same as that for the standard laboratory method. However, rather than running four or more (usually the number is five) specimens through the test and deriving a like number of points, only two tests are run and two points are derived. The first test is with a specimen estimated to be at the optimum moisture content or just on the dry side of the optimum. The second test is with a specimen 2 or 3 percentage points dry of the water content of the first specimen. From these points, and the well documented data on the soil, the optimum moisture content and maximum density can be determined.

(3) Reference: EM 1110-2-1911, 1977.

Limitations: This method depends upon good and well documented data on the local soils. The method also depends upon data which defines a relatively good line of optimums.

Status of the Method: In this method two curves are developed, as opposed to one curve as in the one point method, thus this method can be expected to yield a better estimated curve. As this method provides more accuracy than the one point method, it is perhaps even more applicable to trench backfill compaction process control.

Calibration Procedure: The rammer must be calibrated before initial use and again after each 1,000 molds.

Documentation of Test: Items to be recorded include the mold volumes, specimen masses, and specimen water contents. No example data sheet is provided.

Parameter Measured: Water Content/Density/Compactive Effort

Title of Test Method: Hilf's Rapid

Principle of Test Method: This method determines the optimum moisture content and the maximum density of a soil sample by the standard impact compaction test of three samples, the use of standardized forms, and an estimate of the optimum moisture content.

Test Method

(1) Apparatus: Molds, rammer, extruder, balance, drying oven, sieves.

(2) Procedure: The procedure utilizes the standard impact compaction test. Three samples are tested. The first sample is that at field moisture, a water content estimated to be on the dry side of optimum. The second and third samples are tested at water contents two and four percent more than the field moisture. The derived values are compared to standardized forms and plotted. Without determination of water content, the maximum density and difference of the field moisture from the optimum content can then be determined. This is done by connecting the plotted points, which bracket the optimum moisture content, and thus yield its value.

(3) Reference: Hilf, 1970.

Limitations: This method is only suitable for low plasticity cohesive soils.

Status of the Method: When applicable this method yields sufficient data in much less time than the standard impact tests. However, because it yields only an approximation of the optimum moisture content its applicability is limited.

Calibration Procedure: The rammer must be calibrated before initial use and again after each 1,000 molds.

Documentation of Test: Items to be recorded include the mold volumes, specimen masses, and specimen water contents. No example data sheet is provided.

Method No. 33

Parameter Measured: Water Content/Density/Compactive Effort

Title of Test Method: Ohio Highway Department Nest of Curves

Principle of Test Method: This method determines the optimum moisture content and the maximum density of a soil sample by the standard impact compaction test yielding one point, and its placement on a nest of curves for like soils.

Test Method

(1) Apparatus: Molds, rammer, extruder, balance, drying oven, sieves, circular slide rule.

(2) Procedure: A single test is performed as per one of the standard impact compaction methods. The derived point is then matched to a nest of curves. For field expedience this nest of curves is often contained on a circular slide rule. The optimum moisture content and maximum density are then estimated from the appropriate curve.

(3) Reference: Ohio Highway Department, 1958.

Limitations: This method depends upon good and well documented data on the local soils.

Status of the Method: Where a nest of curves exists this method is a rapid means of determining the optimum moisture content and maximum density. It has proven to be sufficient in a widespread history of usage.

Calibration Procedure: The rammer must be calibrated before initial use and again after each 1,000 molds.

Documentation of Test: Items to be recorded include the mold volumes, specimen masses, and specimen water contents. No example data sheet is provided.

Parameter Measured: Water Content/Density/Compactive Effort

Title of Test Method: Harvard Miniature Compaction

Principle of Test Method: This method determines the optimum moisture content and the maximum density of a soil sample by a kneading compaction test with the Harvard Miniature device.

Test Method

(1) Apparatus: Molds, tamper, extruder, balance, drying oven, sieves.

(2) Procedure: The procedure utilizes different apparatus than does the standard impact compaction test. Also this procedure utilizes kneading as opposed to impact compaction. Other than that, with the exception of specifics, the procedures are quite similar. This procedure is readily adaptable to better represent the actual field compactive efforts to be achieved.

(3) Reference: Wilson, 1970.

Limitations: This method is suitable only for soils passing the No. 4 screen. Thus it is applicable for usage at a hazardous waste disposal facility.

Status of the Method: The method is small in terms of apparatus cost and size, simple, adaptable to duplicate the compactive effort achieved by larger equipment and ideal for field laboratory use. For better representing the actual field compactive efforts to be achieved the kneading action of the Harvard Miniature apparatus is superior to the impact action of the impact devices.

Calibration Procedure: N/A

Documentation of Test: Items to be recorded include the mold volumes, specimen masses, and specimen water contents. No example data sheet is provided.

APPENDIX B

A PARTIAL LIST OF AVAILABLE NUMERICAL MODELS TO DESCRIBE FLOW AND/OR SOLUTE TRANSPORT IN PARTIALLY SATURATED POROUS MEDIA

APPENDIX B: A PARTIAL LIST OF AVAILABLE NUMERICAL MODELS TO DESCRIBE FLOW AND/OR SOLUTE TRANSPORT IN PARTIALLY SATURATED POROUS MEDIA

(Source: Model Annotation Retrieval System database, IGWMC, Holcomb Research Institute, Butler University, Indianapolis, IN)

Ahlstrom, S. W., H. P. Foote, and R. J. Serne. 1976. (Washburn, J. F.) Battelle Pacific Northwest Labs, P.O. Box 999, Richland, WA 99352, MMT-DPRW, To predict the transient three-dimensional movement of radionuclides and other contaminants in unsaturated/saturated aquifer systems.

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Brutsaert, W. 1971. Ingenieurburo Dr. -Ing. Gerhard Bjornsen, Beratender Ingenieur Fur Wasserwirtschaft Und Wasserbau, 54 Koblenz--Kurfurstenstrasse 87A, Fed. Rep. of Germany, 2-D-INFILTRATION MODEL, To compute the temporal and spatial variation of moisture content and water table response in a vertical cross-section of a heterogeneous (layered) anisotropic soil provile, as caused by precipitation, irrigation, and artificial recharge.

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Gupta, S. K., and C. S. Simmons. 1981. Battelle Pacific NW Laboratories, P.O. Box 999, Richland, WA 99352, UNSAT1D, One-dimensional simulation of unsteady vertical unsaturated flow.

Havercamp, R., and M. Vauclin. 1982. Institut De Mecanique De Grenoble, B.P. 53, Centre De Tri, 38041 Grenoble--Cedex, France, SIMTUS, Simulation of one-dimensional, nonsteady flow in unsaturated, isotropic, homogeneous soils.

Hoogmoed, W. 1981. Soil Tillage Laboratory, Agriculture University. Diedenweg 20, Wageningen, The Netherlands, LONG TERM SIMULATION OF UNSATURATED VERTICAL MOISTURE FLOW IN BARE SOILS, To simulate vertical movement of water in the unsaturated zone of a soil profile.

Huyakorn, P.S. 1982. Geotrans, Inc., P.O. Box 2250, Reston, VA 22090, SATURN 2, To study transient, two-dimensional variably saturated flow and solute transport in anisotropic, heterogeneous porous media.

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Narasimhan, T. N. 1981. (Cole, C.R.), Battelle Pacific NW Laboratory, Water and Land Resources Division, P. O. Box 999, Richland, WA 99352, TRUST, to compute steady and nonsteady pressure head distribution in multidimensional, heterogeneous, variably saturated, deformable porous media with complex geometry.

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Sagar, B. 1982. Analytic and Computational Research, Inc., 3106 Inglewood Boulevard, Los Angeles, CA 90066, VADOSE, Steady or transient, two-dimensional, areal, cross-sectional or radial simulation of density-dependent transport of moisture, heat and mass in variably saturated, heterogeneous, anisotropic porous media.

Sagar, B. 1982. Analytic and Computational Research, Inc., 3106 Inglewood Boulevard, Los Angeles, CA 90066, FLOTRA, Steady or transient, two-dimensional, areal, cross-sectional or radial simulation of density-dependent flow, heat, and mass transport in variably saturated, anisotropic, heterogeneous deformable porous media.

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Yeh, G. T., and D. S. Ward. 1981. Environmental Sciences Division, Oak Ridge National Laboratory, Oak Ridge, TN 37830, FEMWATER/FECWATER, A two-dimensional model to simulate transient, cross-sectional flow in saturated-unsaturated anisotropic, heterogeneous porous media.

APPENDIX C

PROGRAM LINERSOL: MODIFIED VERSION OF PROGRAM SOILINER

APPENDIX C

PROGRAM LINERSOL: MODIFIED VERSION OF PROGRAM SOILINER

The computer program SOILINER written by Dan Goode of GCA (1983) solves the equation of flow in an unsaturated media and describes the infiltration in a partially saturated liner. The program determines the infiltration rate and moisture content profiles with time and thus determines the time taken by a liner system to achieve a steady-state moisture content profile after establishment of a liquid head on top of the liner. It may be necessary to solve the solute transport equation (C.2) to determine the appropriate transit time. The program SOILINER was modified for this purpose to include the solute transport equation by a simple finite difference formulation.

Several computer programs are available to solve the flow and solute transport equations for a partially saturated porous media in general. These programs, however, solve the equations separately; it is necessary to solve the flow equation first and store the variable values required by the transport equation at all flow time steps to obtain the solution to the solute transport equation. A program that solves both equations simultaneously will be easier to use because there will be no need to save the intermediate variables. The advantage of the program SOILINER is that it is written specifically for a liner system and is thus easy to use, although it solves only the flow equation. Therefore, in the present work, the program SOILINER was modified to include a simultaneous solution of the solute transport equation.

C.1 METHODOLOGY

The vertical transport of a dissolved solute chemical species in aqueous phase in a liner is described by the solute transport equation, expressed by Equations C.1 and C.2. In terms of moisture content, θ, and effective hydraulic conductivity, K, Equations C.1 and C.2 may be written as:

$$\frac{\partial}{\partial t}(R\theta C) = \frac{\partial}{\partial z}\left(D\frac{\partial C}{\partial z}\right) - \frac{\partial}{\partial z}(vC) \text{ , and} \qquad (C.1)$$

$$v = -K\left(\frac{\partial \Psi}{\partial z} - 1\right) . \qquad (C.2)$$

The dispersion coefficient D includes both the molecular diffusion coefficient of the species of interest and the hydrodynamic dispersion coefficient and may be expressed by (Huyakorn and Pinder, 1983):

$$D = \frac{\alpha_L v}{\Phi} + D^o \quad , \tag{C.3}$$

where α_L is the longitudinal dispersivity, v is the Darcian velocity given by Equation (C.2), D^o is the molecular diffusion coefficient, and Φ is porosity.

Since the variables θ, v, or pressure head, Ψ, depend upon the solution of flow equation, it is necessary to solve the flow equation first by advancing its solution by a flow time step, Δt_{flow}. If the values of moisture content, θ, and pressure head, Ψ, are assumed to be constant during the flow time step, Equation (C.1) may be rewritten as:

$$R\theta \frac{\partial C}{\partial t} = D \frac{\partial \left(\frac{\partial C}{\partial z}\right)}{\partial z} + \frac{\partial\, KC\left(\frac{\partial \Psi}{\partial z} - 1\right)}{\partial z} \quad . \tag{C.4}$$

Equation (C.4) may now be expressed by a simple explicit forward time space-centered finite difference scheme, denoting i for a nodal position z in liner, j as time level, Δz_i as nodal increment at node i, and Δt as time increment (the nodal grid system is shown in Figure C-1).

$$R_i \theta_i \frac{C_i^{j+1} - C_i^j}{\Delta t} = D_i \left[\frac{\left(\frac{C_{i+1}^j - C_i^j}{\Delta z_i}\right)\left(\frac{C_i^j - {}_iC_{-1}}{\Delta z_{i-1}}\right)}{\frac{1}{2}(\Delta Z_i + \Delta z_{i-1})} \right]$$

$$+ \frac{K_i C_i^j \left(\frac{\Psi_{i+1} - \Psi_i}{\Delta z_i} - 1\right) - K_{i-1} C_{i-1}^j \left(\frac{\Psi_i - \Psi_{i-1}}{\Delta z_{i-1}} - 1\right)}{\frac{1}{2}(\Delta Z_i + \Delta z_{i-1})} \quad . \tag{C.5}$$

Equation (C.5) is explicitly formulated, so the stability of its solution depends upon the time increment, Δt. When stability criteria are applied, an upper limit on the time increment, Δt, may be set as:

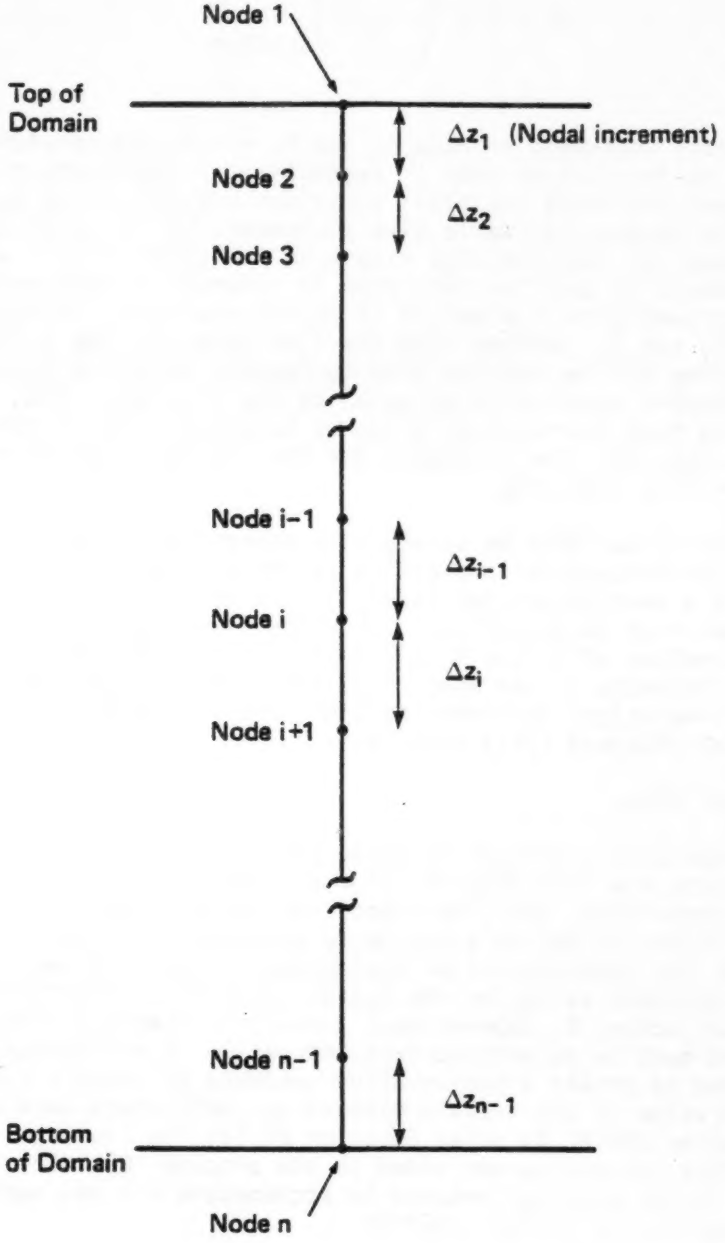

Figure C-1. Mesh-centered nodal grid system.

$$\Delta t \leq 0.5 \left(\frac{\Delta z_i^2 \ \theta_i}{D_i} \right)_{\text{minimum}}, \qquad \text{(C.6)}$$

where Δz_i is nodal increment at node i, and θ_i and D_i are moisture content and dispersion coefficient at node i, respectively. Since the right-hand side depends upon the nodal position, a minimum possible value must be determined. The maximum allowable time increment, Δt, as given by Equation (C.6), may be smaller than the flow time step, in which case it will be necessary to subdivide the flow time step in a number of smaller appropriate time increments for the solute transport equation. In such a case, the variables θ_i and Ψ_i, derived from the flow equation, may be assumed to be constant during all the smaller time increments until the solution of the solute transport equation is advanced by the flow time step, Δt_{flow}. If the allowable time increment as given by Equation (C.6) is greater than the flow time step, the time increment for the transport equation is set the same as the flow time step.

Equation (C.5) may thus be solved with appropriate time increments, Δt, to obtain the concentration profiles at the next time level. To obtain the solution at a particular time level, j, the moisture content and pressure head values must be specified. Since the flow equation is solved first, two set values of θ_i and Ψ_i are available at time levels j and j+1, respectively. Attempts to use mean or j-level values of θ_i and Ψ_i were not successful in simulations described earlier, however, stable solutions were obtained through advanced (j+1) time level values.

C.2 SUBROUTINE CONCAL

The computational procedure to solve Equation (C.5) is outlined in Figure C-2 showing the flow diagram. For clay materials of low saturated hydraulic conductivities, the flow velocities are considerably smaller, and the molecular diffusion may be expected to dominate the dispersion process. In such a case, the contribution of hydrodynamic dispersion may be ignored by assuming a constant value for the overall dispersion coefficient, D. The retardation factor, R, depends upon a specific chemical-liner material combination and must be determined experimentally. A noninteractive species (R=1) is assumed to obtain a conservative estimate of transit time. R=1 and a constant value of the overall dispersion coefficient were used to write a subroutine CONCAL to solve Equation (C.5); its listing is given in Figure C-3. This subroutine was added to the program SOILINER with additional minor changes in the original program to accommodate the new subroutine. The modified version is called LINERSOL.

C.3 PROGRAM LINERSOL SIMULATIONS

This program was used to carry out the solute transport simulations described earlier. In all these numerical simulations, a liquid head of 30 cm of water was assumed to be established at the top of the liner. The

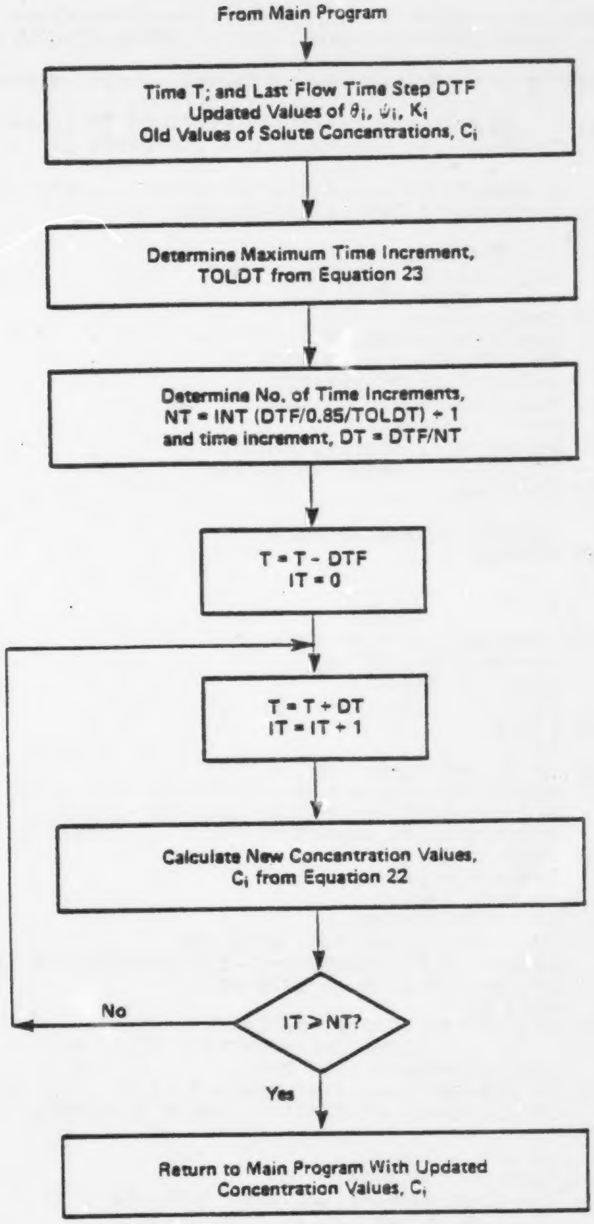
From Main Program
Time T; and Last Flow Time Step DTF
Updated Values of θ_i, ψ_i, K_i
Old Values of Solute Concentrations, C_i
Determine Maximum Time Increment,
TOLDT from Equation 23
Determine No. of Time Increments,
NT = INT (DTF/0.85/TOLDT) + 1
and time increment, DT = DTF/NT
T = T - DTF
IT = 0
T = T + DT
IT = IT + 1
Calculate New Concentration Values,
C_i from Equation 22
IT ≥ NT?
No
Yes
Return to Main Program With Updated
Concentration Values, C_i

Figure C-2. Computational flow diagram of subroutine CONCAL.

```
C***********************************************************************
      SUBROUTINE CONCAL(PSI,RMOIST,STARK,DZ,CONC,NUMEL,DT,ICON,DFUZVT,
     *                  TOLDT)
C***********************************************************************
C
C.....CALCULATES CONCENTRATION PROFILES OF LEACHATE SPECIES DUE
C.....TO ADVECTION AND MOLECULAR DIFFUSION OF THE SPECIES
C
      DIMENSION PSI(1),RMOIST(1),STARK(1),DZ(1),CONC(1),CONCN(200)
      DIMENSION A(200),B(200),C(200)
C
      IF(ICON.NE.0) GOTO 20
      CONC(1)=1.0
      DO 10 I=2,NUMEL
   10 CONC(I)=0.0
      CONC(NUMEL+1)=0.0
      IF (DFUZVT.LE.0.0) GOTO 17
      VMIN=DZ(1)*DZ(1)*RMOIST(1)
      NUMNP=NUMEL+1
      DO 15 I=2,NUMNP
      PROD=DZ(I-1)*DZ(I-1)*RMOIST(I)
      IF (PROD.LT.VMIN) VMIN=PROD
   15 CONTINUE

      TOLDT=0.425*VMIN/DFUZVT
   17 ICON=1
C
      RETURN
C
   20 IF(DFUZVT.LE.0.0) GOTO 25
      NT=INT(DT/TOLDT)+1
      DTN=DT/FLOAT(NT)
      GOTO 30
   25 DTN=DT
      NT=1
   30 DO 35 I=2,NUMEL
      A(I)=STARK(I-1)*(1.0+(PSI(I)-PSI(I-1))/DZ(I-1))
      B(I)=STARK(I)*(1.0+(PSI(I+1)-PSI(I))/DZ(I))
      C(I)=RMOIST(I)*ABS(DZ(I)+DZ(I-1))/DTN/2.0
   35 CONCN(I)=0.0
      CONCN(NUMEL+1)=0.0
      DO 150 IT=1,NT
      DO 100 I=2,NUMEL
      IF(CONC(I-1).LE.0.0) GOTO 100
      CONCN(I)=(A(I)*CONC(I-1)-B(I)*CONC(I))/C(I)+CONC(I)
      IF (DFUZVT.GT.0.0) GOTO 40
      GOTO 50
   40 D=DFUZVT*((CONC(I)-CONC(I-1))/DZ(I-1) -
     *            (CONC(I+1)-CONC(I))/DZ(I) )
      CONCN(I)=CONCN(I) + D/C(I)
   50 IF (CONCN(I).LE.0.0) CONCN(I)=0.0
      IF (CONCN(I).GT.CONC(1)) CONCN(I)=CONC(1)
  100 CONTINUE
      DO 120 I=2,NUMEL
  120 CONC(I)=CONCN(I)
      IF (CONC(NUMEL-1).LE.0.0) GOTO 150
      IF (CONC(NUMEL).LE.0.0) GOTO 150
      CONC(NUMEL+1)=CONC(NUMEL)*CONC(NUMEL)/CONC(NUMEL-1)
  150 CONTINUE
C
      RETURN
      END
```

Figure C-3. Listing of the subroutine CONCAL in program LINERSOL.

liner was assumed to be 200 cm thick with a saturated hydraulic conductivity of 1 × 10^{-7} cm/s. At the bottom of the liner a negative pressure head of -500 cm was assumed to exist due to the influence of the underlying permeable site soil of 5 m depth above the water table. Figures C-4 through C-6 show the advancement of moisture content profiles as determined by program SOILINER for three initial saturations (50 percent, 80 percent, and 95 percent) of liner. Figures C-7 through C-9 show the solute concentration profiles in the liner at different times obtained by using only advective transport mechanism (D=0) in program LINERSOL for the same three initial liner saturations. Figures C-10 through C-12 show the advancement of concentration profiles with time due to both advection and molecular diffusion using program LINERSOL. The dispersion coefficient was assumed to be constant and equal to 1 × 10^{-6} cm^2/s. These figures clearly show that the transit times based on solute transport can differ from those based on liner saturation and that transit times for given criteria depend upon initial liner saturation.

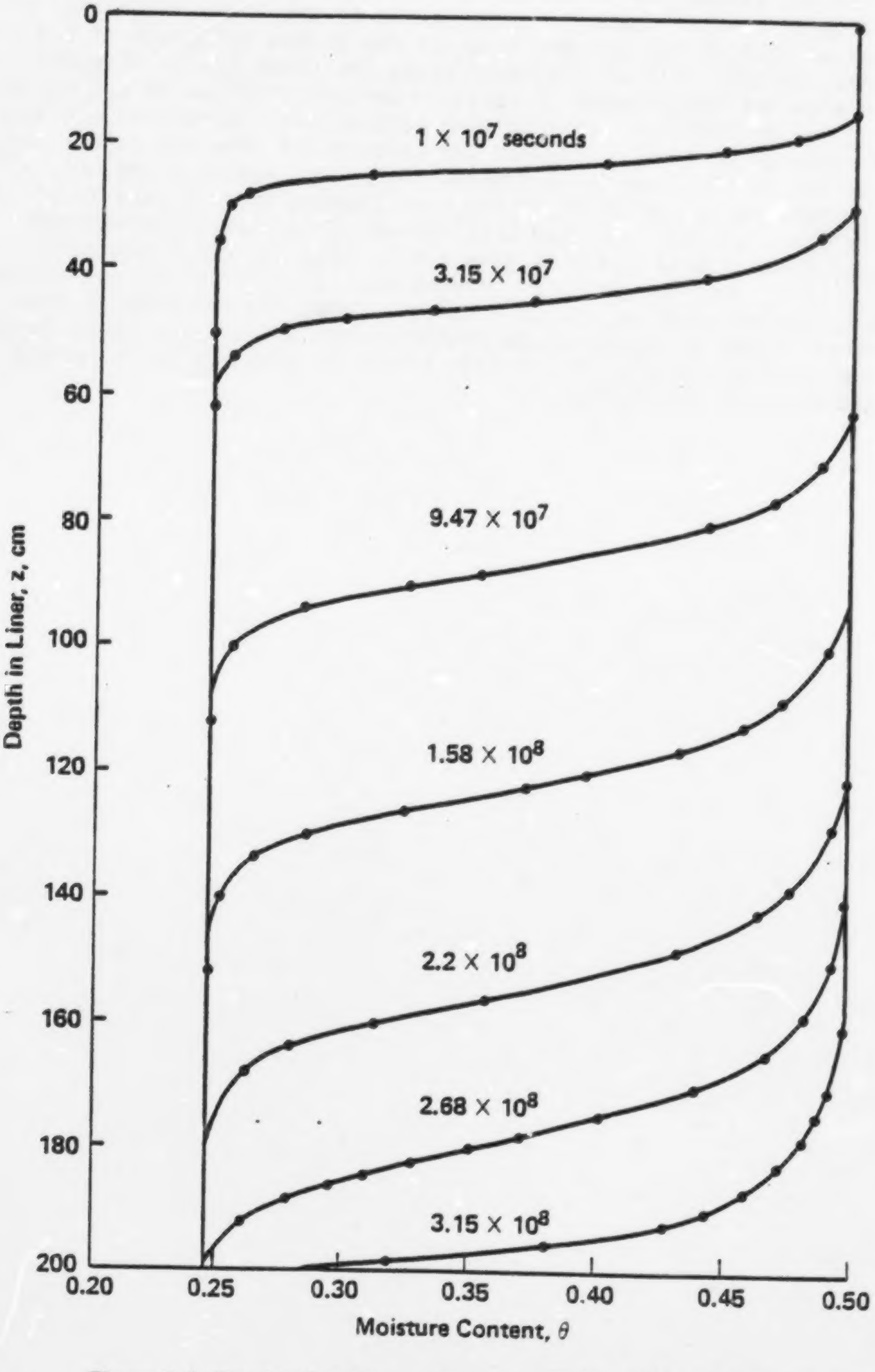

Figure C-4. Simulated moisture content profiles in a liner at different times, for initial liner saturation of 50 percent.

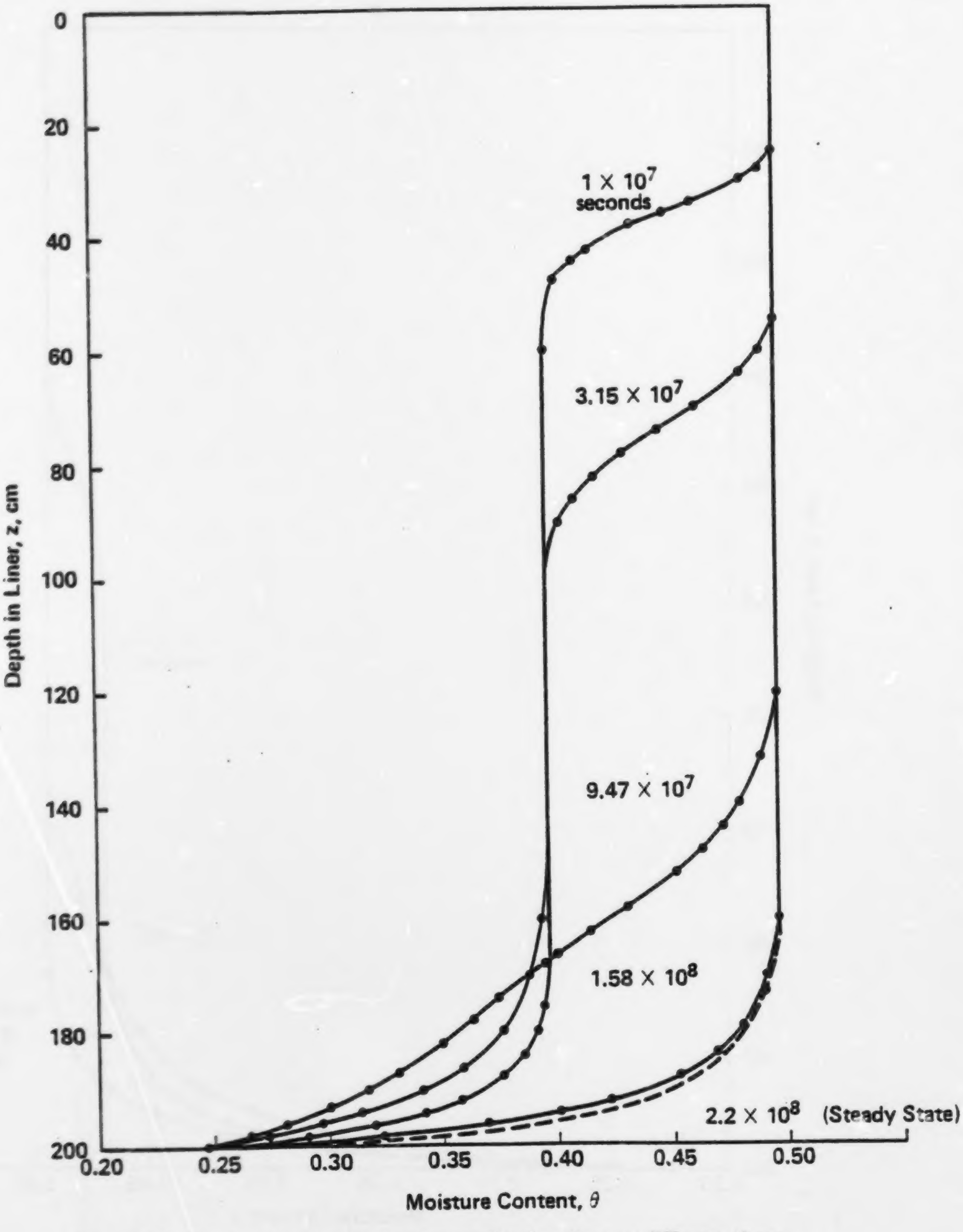

Figure C-5. Simulated moisture content profiles in a liner at different times, for initial liner saturation of 80 percent.

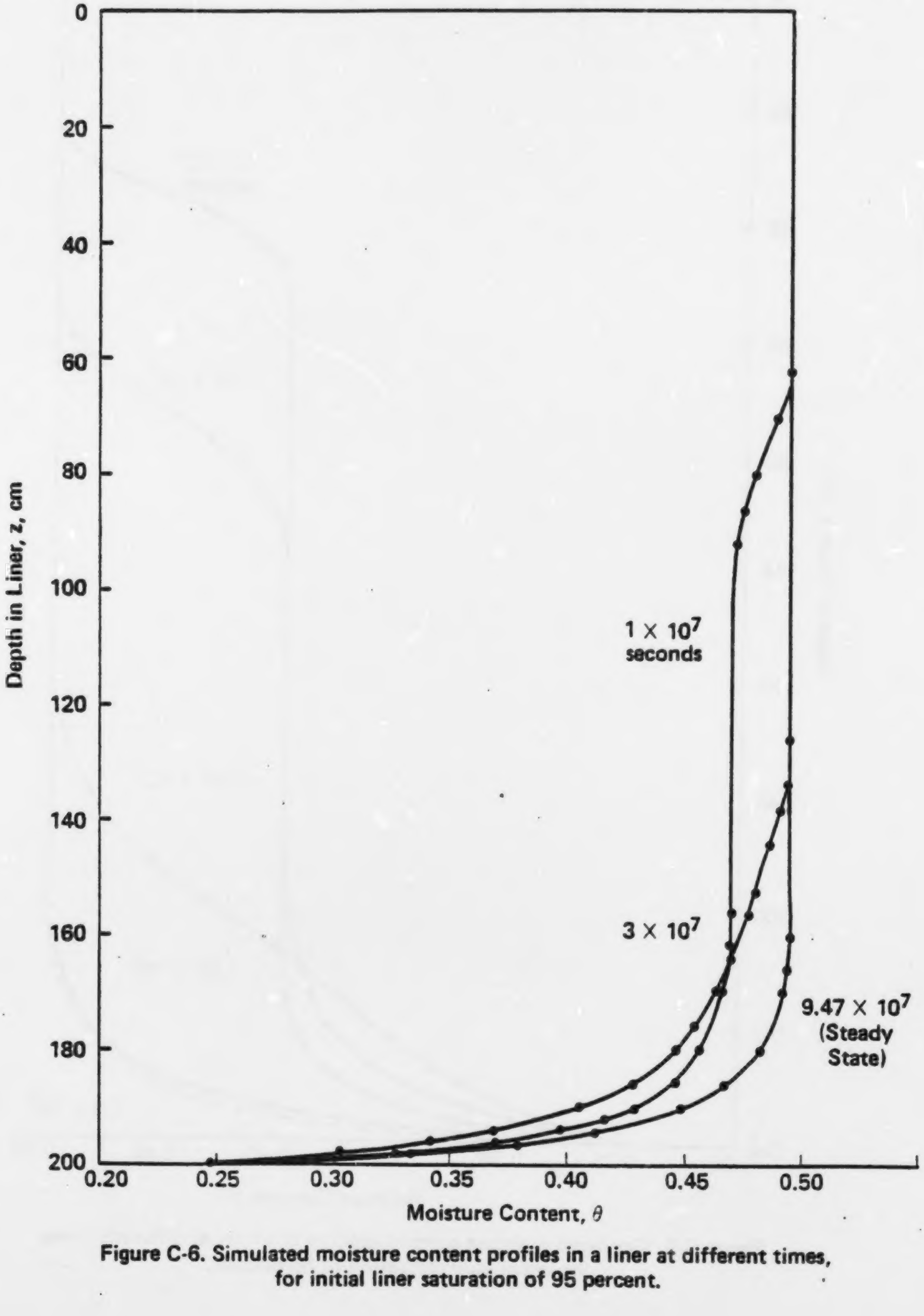

Figure C-6. Simulated moisture content profiles in a liner at different times, for initial liner saturation of 95 percent.

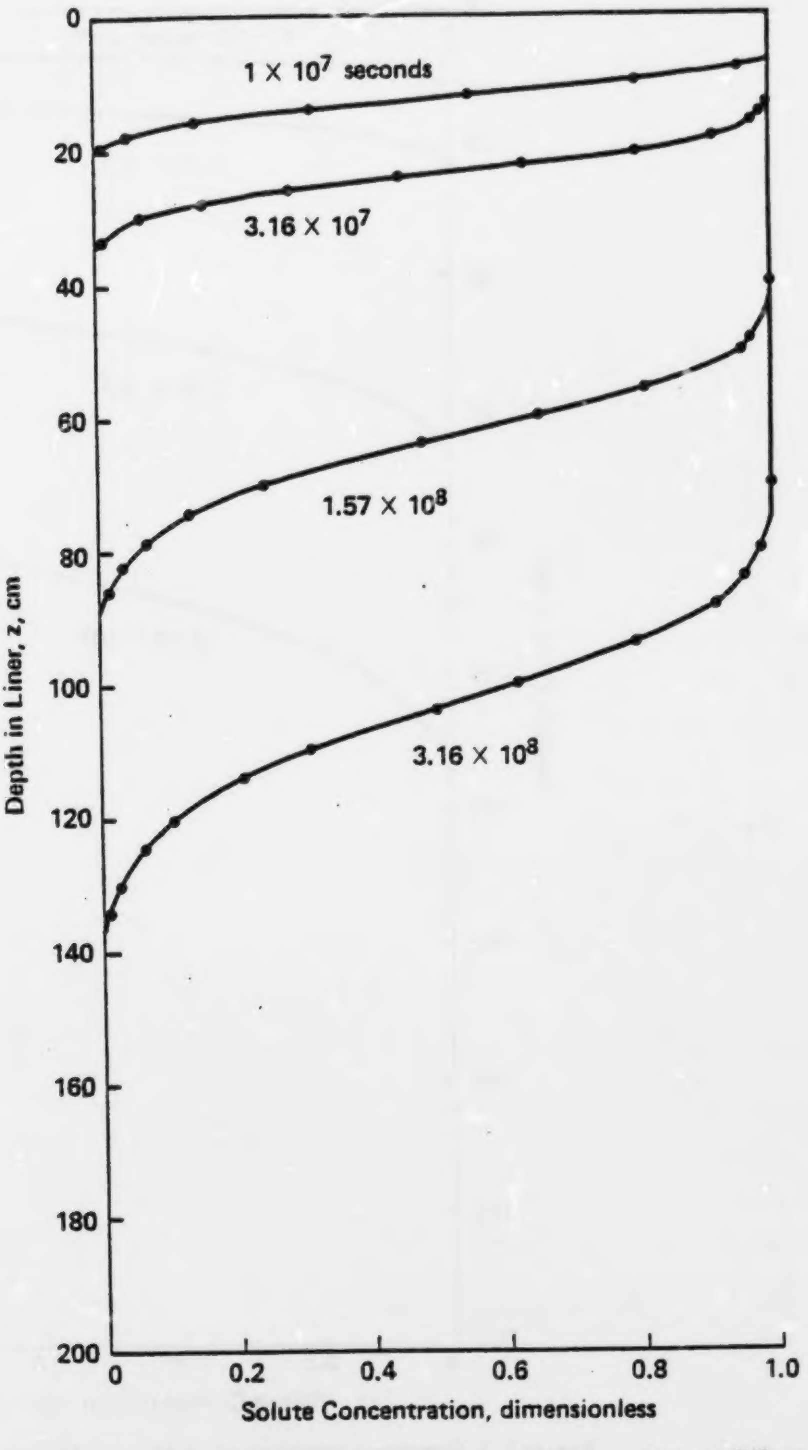

Figure C-7. Simulated solute concentration profiles in a liner at different times, for initial liner saturation of 50 percent (advection only).

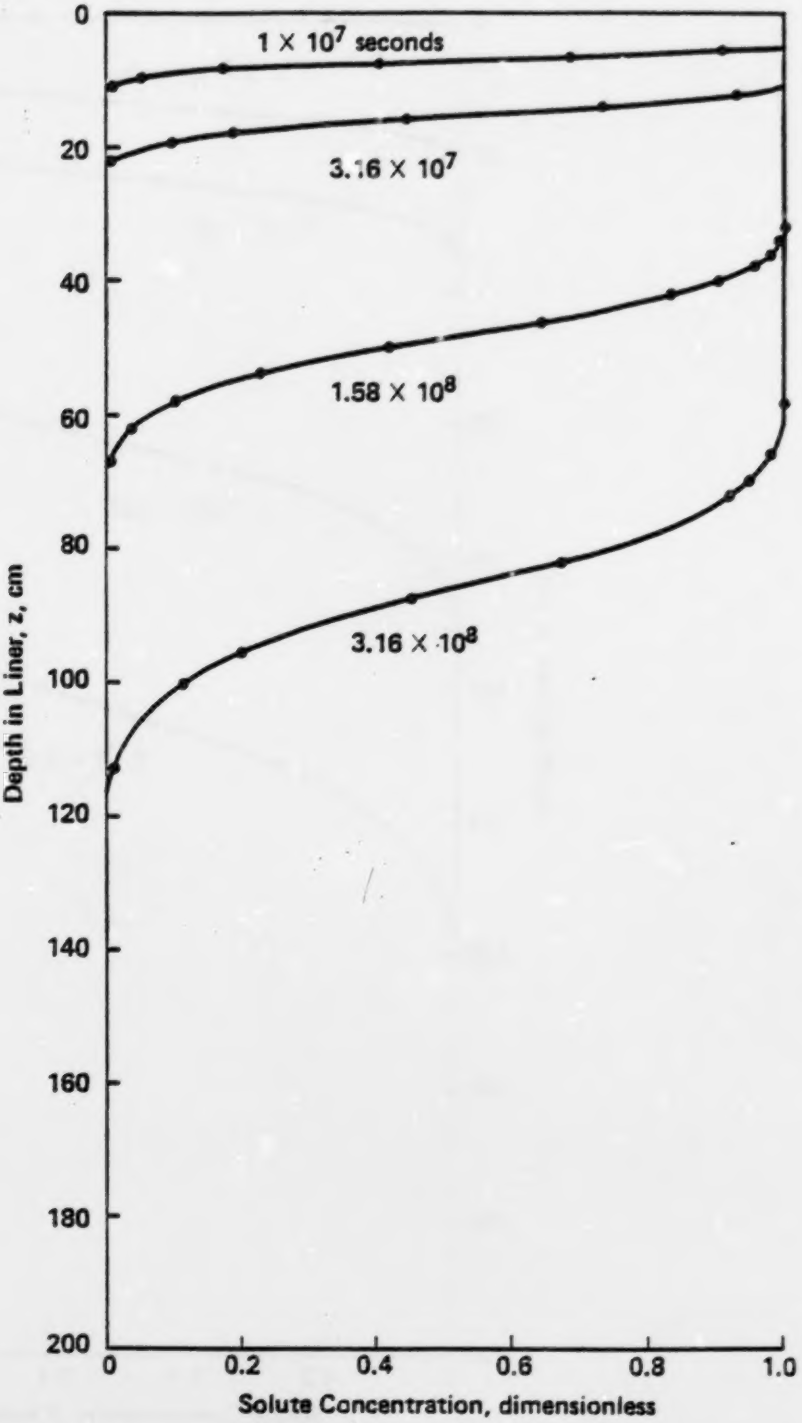

Figure C-8. Simulated solute concentration profiles in a liner at different times, for initial liner saturation of 80 percent (advection only).

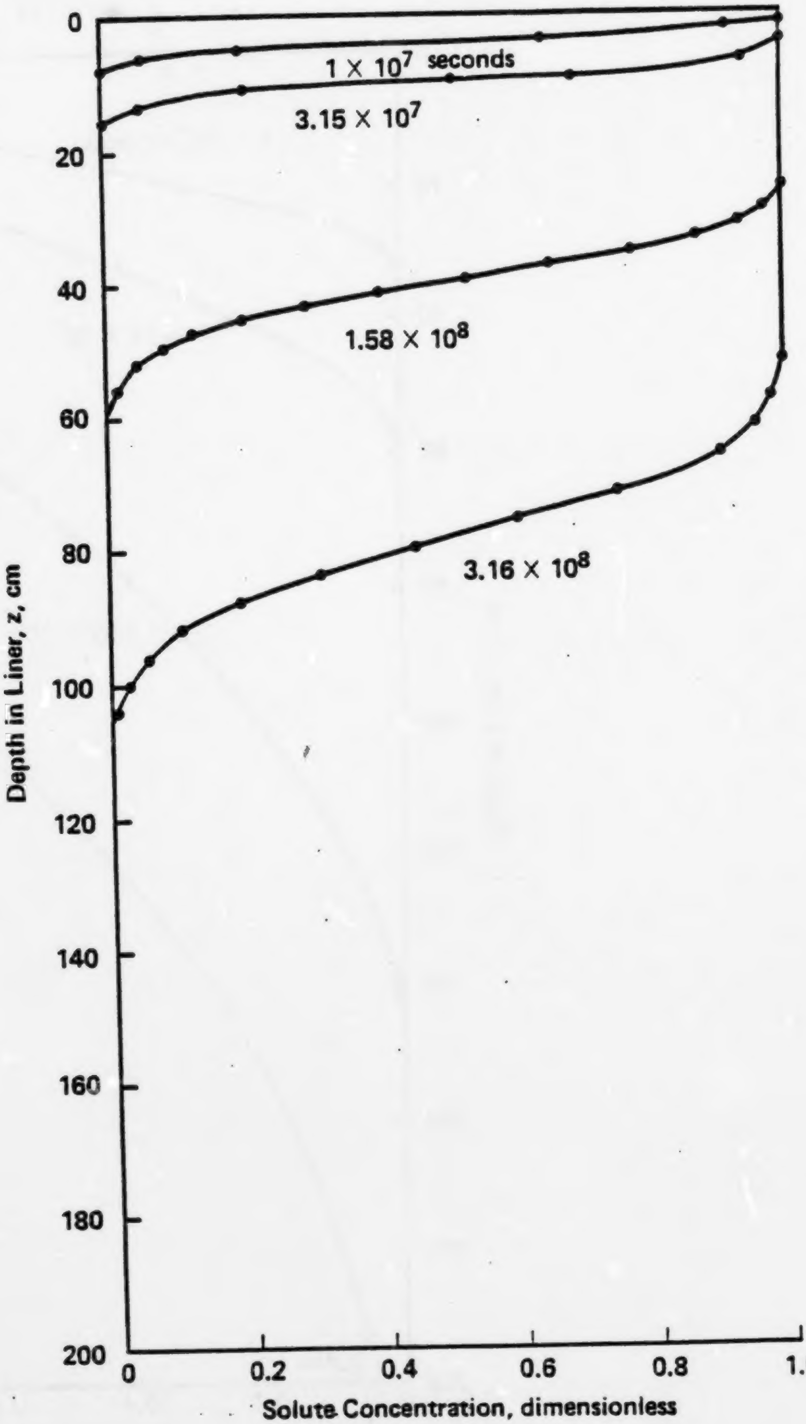

Figure C-9. Simulated solute concentration profiles in a liner at different times, for initial liner saturation of 95 percent (advection only).

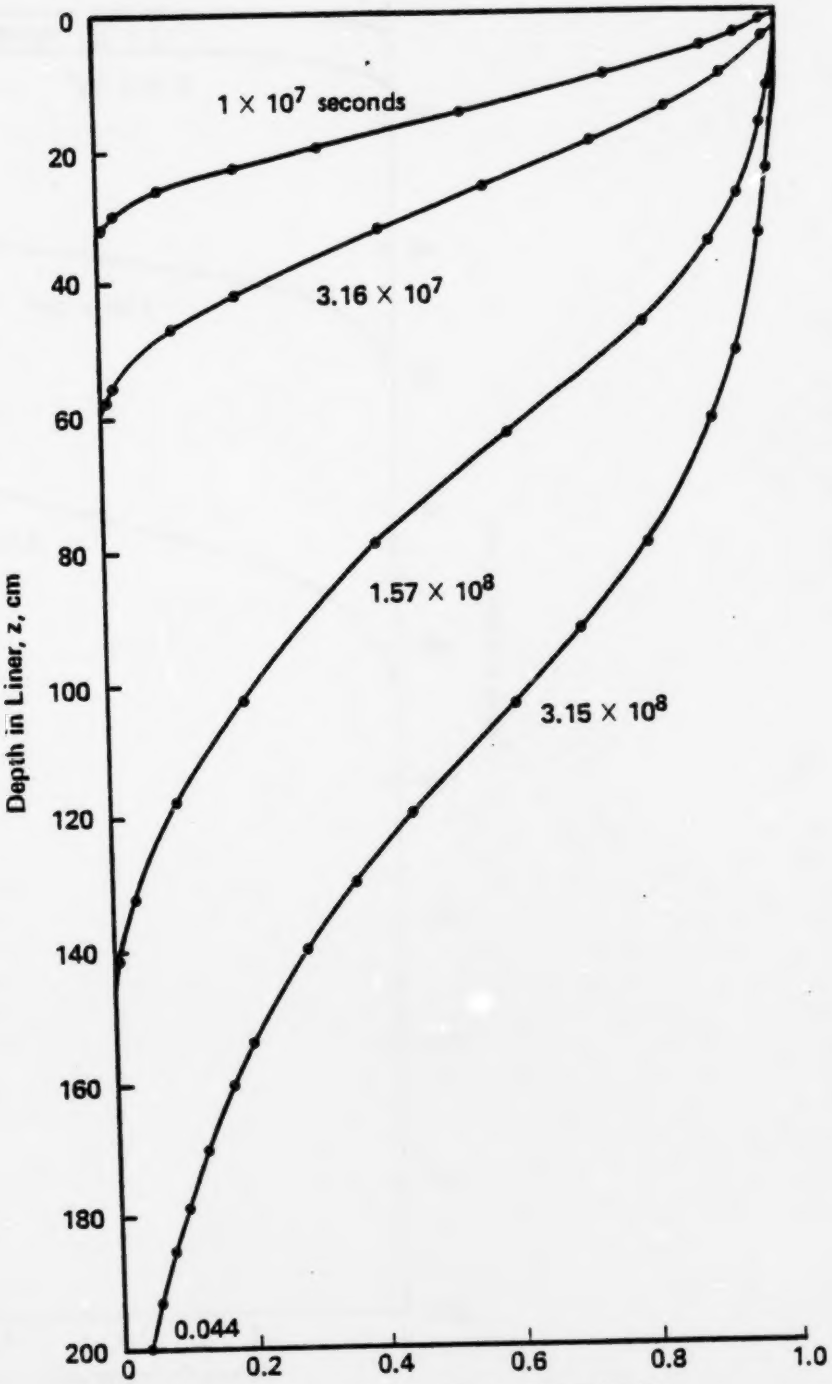

Figure C-10. Simulated solute concentration profiles in a liner at different times, for initial liner saturation of 50 percent (advection and molecular diffusion).

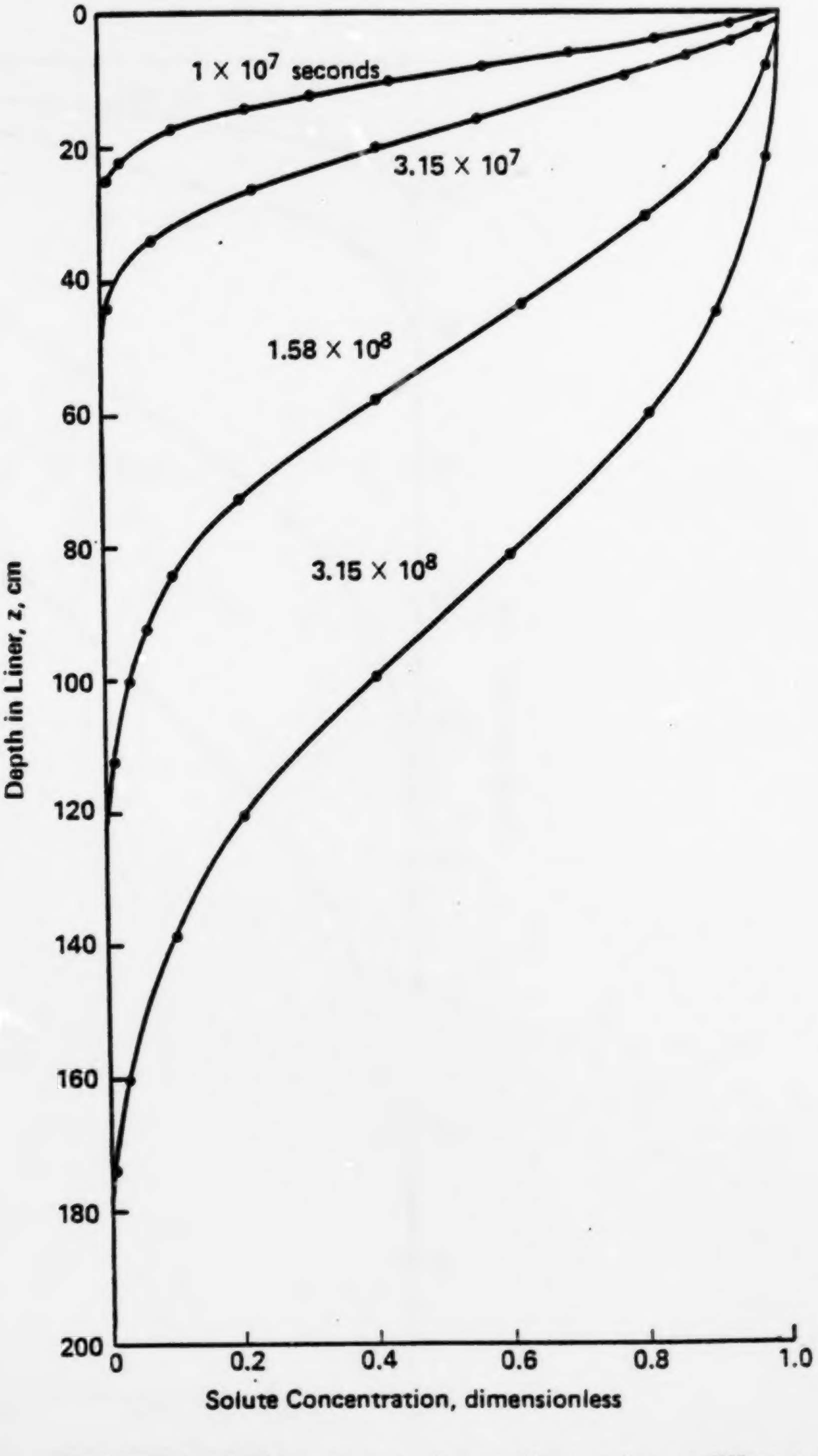

Figure C-11. Simulated solute concentration profiles in a liner at different times, for initial liner saturation of 80 percent (advection and molecular diffusion).

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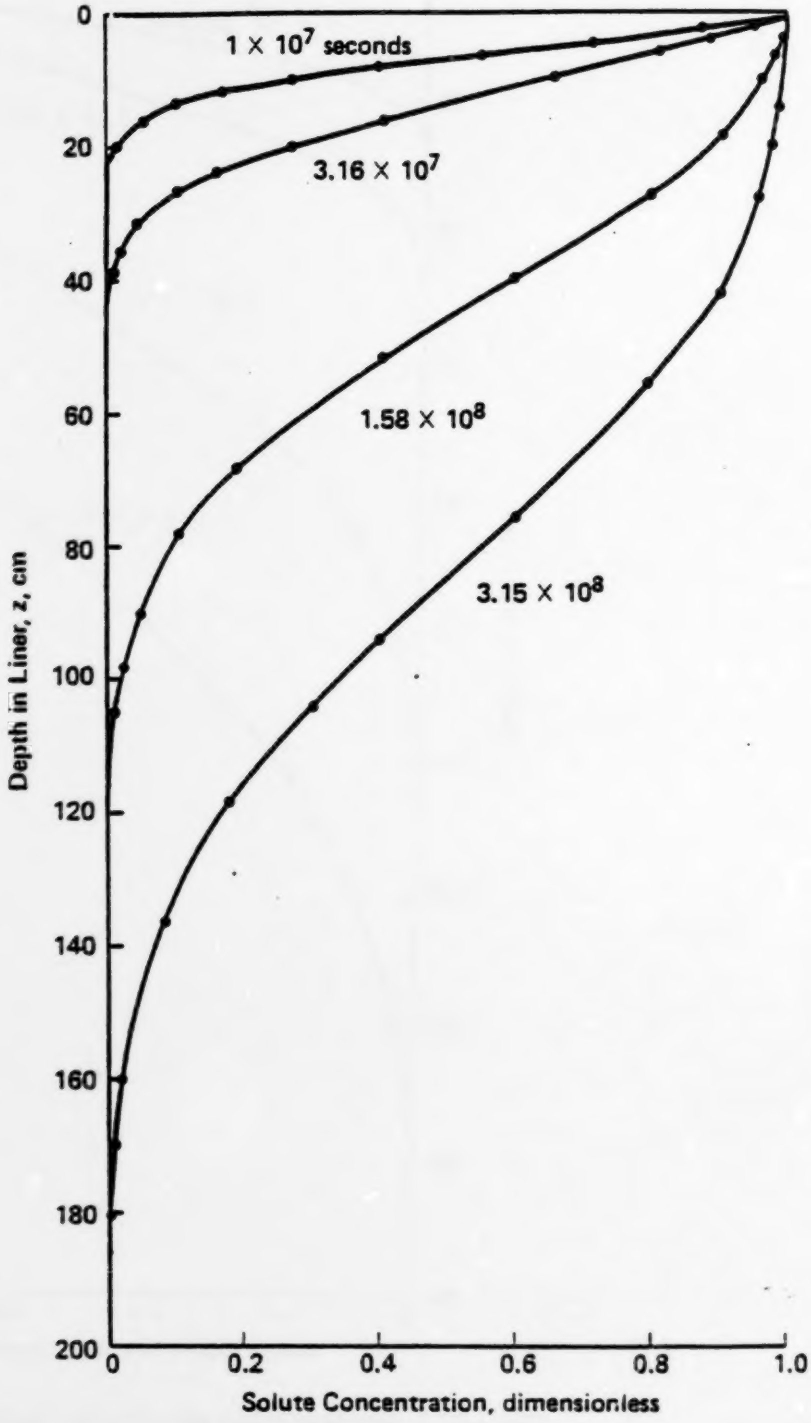

Figure C-12. Simulated solute concentration profiles in a liner at different times, for initial liner saturation of 95 percent (advection and molecular diffusion).